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ARMY ENGINEERING DISTRICT NORFOLK VA
NATIONAL DAM SAFETY PROGRAM. TALBOTT DAM NUMBER (VA 14101), PAT--ETC(U)
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Name Of Dam: TALBOTT DAM

Location: PATRICK COUNTY

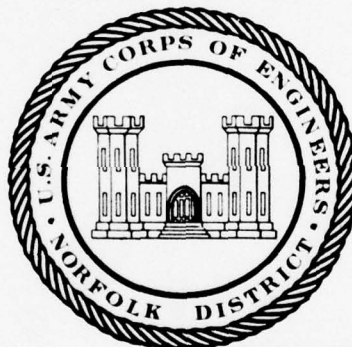
Inventory Number: VA 14101

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

AD A063581

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

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TABLE OF CONTENTS

Brief Assessment of Dam	1
Overview Photo	3
Section 1: PROJECT INFORMATION	4
Section 2: ENGINEERING DATA.	8
Section 3: VISUAL INSPECTION	11
Section 4: OPERATIONAL PROCEDURES	13
Section 5: HYDRAULIC/HYDROLOGIC DESIGN	15
Section 6: DAM STABILITY	18
Section 7: ASSESSMENT/REMEDIAL MEASURES	20

Plates

Appendix I:	Maps and Drawings
II:	Photographs
III:	Field Observations
IV:	Geology Reports
V:	Underwater Inspection Report
VI:	Contract Specifications (Selected Pages)
VII:	References

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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Talbott Dam

State: Virginia

County: Patrick

Coordinates: 3640.6 8023.8

Stream: Dan River

Date of Inspection: 27 July 1978

Talbott Dam is an unreinforced concrete arch dam, 143.5 feet high, and 510 feet long at the crest, sited in a deep gorge at the head waters of the Dan River. Design calculations are not available. Searches by both the designer and the owner were unsuccessful in locating any design calculations. The Geology report and construction drawings indicating concrete strength and foundation grouting programs were reviewed and found to be adequate.

The dam is in good condition. Seepage has occurred through an estimated 30 to 40% of all construction and monolith joints leaving calcium deposits on the face at the dam. A 1/2-inch horizontal crack, 30 feet long and 4 lifts from the top of the dam was found starting at the contact between the left abutment and downstream face of the dam. Seepage through the crack was estimated to be 1 gallon per minute. Based on visual observation, the crack does not warrant serious concern at this time, however, further investigation is required.

Seepage was observed on the left abutment in several areas and the largest seep was located approximately 50 feet below the top of the dam and 150 feet downstream. The seepage on the left abutment is not a problem at this time, however, volume of water should be monitored. (See Section 7).

The probable maximum flood would overtop the dam (top of parapet wall) by 6.4 feet, reaching elevation 2549.9 with flashboards removed. The design drawings indicate a flood level of elevation 2538, a difference of 11.9 feet. This is cause for concern since no structural analysis is available. Analysis of dam stability for this dam is beyond the scope of a Phase I report.

With the flashboards removed the spillway passes only 35% of the PMF. Since the spillway passes less than $\frac{1}{2}$ PMF and the stability is unknown, the spillway is considered seriously inadequate. If structural analysis proves that the dam is stable under critical loading conditions, the spillway may be considered merely inadequate in lieu of seriously inadequate.

A dam with a seriously inadequate spillway is considered to be unsafe. Considering Talbott Dam to be unsafe for overtopping conditions is a conservative assumption since essentially the stability of the dam is unknown. However, this approach is consistent with good engineering judgement since the lives of many people are affected. This should be considered an interim status until the stability of the dam can be determined by structural analysis.

It is recommended that the owner, at his expense, take the following action:

a. Establish an effective warning system to provide evacuation of the downstream flood plain (downstream of Townes Dam) if necessary. The warning system should be operable until it can be shown by analysis that Talbott Dam is stable with appropriate safety factors included under critical loading conditions. It must be considered that Talbott Dam is threatened whenever the reservoir level exceed 2538, until analysis shows otherwise.

b. Engage the services of a Professional Engineer to perform a structural stability analysis under all loading conditions. The 1/2" crack at the left abutment should be evaluated when the structural investigation is accomplished. Structural analysis should be completed within 6 months.

c. Engage the services of a professional consultant to complete a report of remedial mitigating measures which will correct the seriously inadequate spillway and have an agreement with the Commonwealth of Virginia to a reasonable time frame in which all remedial measures will be complete. This work shall be completed within six months from the date of notification to the Governor.

d. In addition the following recommendations described in Section 7 should be performed: (1) remove flashboards, (2) inspect stop gates, (3) inspect siltation, (4) verify maintenance of mechanical equipment, (5) relocate transformers, fuse cutouts and surge arrestors, (6) repair transformer oil leakage, (7) install emergency lighting, (8) inspect left abutment, (9) monitor seepage.

Submitted By:
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Chief, Design Branch

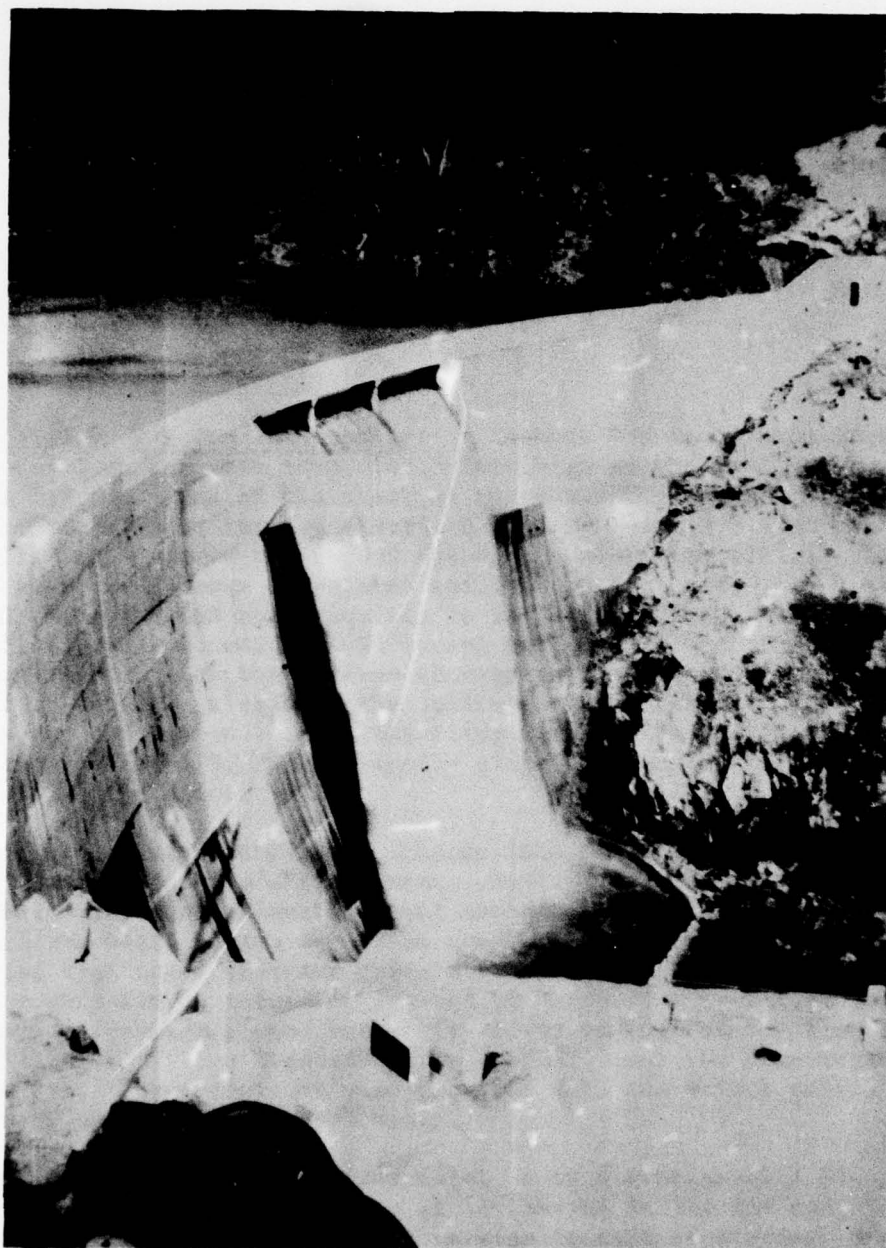
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Chief, Engineering Division

SEP 27 1978



TALBOTT DAM
(PHOTO PRIOR TO 1940)

TALBOTT DAM

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 Aug 72 authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose of the Phase I inspections is to identify expeditiously those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Talbott Dam is an unreinforced, 280 feet radius concrete arch, 143.5 feet high at the spillway section, and 510 feet long at the crest of the dam. The top of the dam consists of a walkway at elevation 2540 MSL with 3.5 feet parapet walls which extend the top of the dam to elevation 2543.5. The walkway passes over an ungated spillway 80 feet wide with a crest of 2532 MSL, located approximately in the center of the dam. The spillway passes excess flows during periods of high streamflow. Flashboards raise the maximum storage pool three feet to elevation 2535. Water passing over the spillway strikes the downstream face of the dam and washes into the stilling basin which has an apron elevation of 2409. Symmetrically placed wingwalls direct flows into a 40 foot wide stream bed.

The intake for controlled discharge into the stilling basin is located 239 feet from the right abutment. Water flows from the reservoir through a 36-inch diameter steel conduit that discharges into the stilling basin of the dam. The conduit intake at the dam includes a fixed type trash rack and guide slot for an intake stop gate. The stop gate can be raised or lowered by means of a manually operated winch. Tandem-arranged butterfly valves to control the flow to the stilling basin are located in the conduit, near the downstream face of the dam. The conduit is also provided with two 6-inch valved by passes and a 6-inch valved drain.

The centerline of the conduit inlet is at elevation 2413 MSL. The conduit is 6 feet in diameter at its intake in the dam and, in a conical transition, is reduced in size to that of a 3-foot diameter pipe. It is this steel pipeline that incorporates the flow control valves, and their valved bypasses. The line is extended to the underside of the stilling basin where it is connected to a 20-foot diameter discharge ring which is made of 24-inch diameter cast iron

pipe. The ring includes twenty-six equally spaced 5 1/8-inch diameter vertical nozzles which penetrate the stilling basin concrete apron, at elevation 2409 forming a fountain type energy dissipator.

The main flow control valves for the conduit consist of tandem-arranged 36-inch diameter butterfly valves (Chapman-Crane). The service (downstream) valve is equipped with a motorized geared drive operator which is locally operated from controls within the valve house. Remote operation from the power plant is no longer available due to failure of the control cable. In the event of an electrical malfunction, this valve can be operated manually. The emergency valve is similar to the service valve with the exception that it must be manually operated. The emergency valve is normally fully open. The service valve is normally closed, and is opened only when replenishment of the Townes Dam reservoir is required. Both Talbott and Townes Dams are part of the Pinnacles Hydro-Electric Project. Talbott Dam provides most of the storage for the project while the pipeline to the hydro station begins at Townes Dam.

Duplicate 6-inch bypasses, complete with tandem-arranged normally open gate valves, are provided to achieve limited flow, while either or both 36-inch valves are closed.

In addition to the bypasses, there is provided upstream from the 36-inch emergency valve, a single 6-inch line, with a normally closed gate valve, that discharges directly into the stilling basin. This line is evidently provided in the event that limited flow to the stilling basin may be required while the valves in the 36-inch line and in its 6-inch bypasses are closed for maintenance purposes.

The stop gate for the intake serves to close the intake entrance when major maintenance is required on the 36-inch butterfly valves, the 6-inch bypass valves, and the 6-inch drain valve. The gate is permanently suspended by means of stainless steel wire rope from a crank-operated, base-mounted, geared-drum, winch unit. The winch is mounted on structural steel supports located just above the top of the dam walkway. The supports project beyond and are mounted in bracket fashion on the upstream face of the dam. The housing for the winch unit is of the open-sided roofed type.

An examination of the contract drawings did not reveal any details of the gate, but it is speculated that it is a conventional type fabricated of structural steel shapes and plate with rubber seals.

1.2.2 Location: Talbott Dam is located on the headwaters of the Dan River 4 miles south of the meadows of Dan where U.S. 58 crosses the Blue ridge parkway in southwestern Virginia.

1.2.3 Size Classification: The dam is classified large due to its height of 143.5 feet and storage capacity of 9600 acre feet.

1.2.4 Hazard classification: The dam is located 5½ miles upstream of the Townes Dam and 9 miles from the Kibler Valley. Failure of Talbott Dam would release more than 8,000 acre-feet of water into Townes Reservoir which has a capacity of 1377 acre-feet under normal operating conditions. This would probably produce a failure of Townes Dam and inundate the Kibler Valley resulting in a loss of many lives. In accordance with the "Recommended Guidelines for Safety of Dams" published by the Office of the Chief of Engineers for dams, the dam is given a high hazard classification. Hazard classification used to categorize dams is a function of location only and unrelated to the stability or probability of failure.

1.2.5 Ownership: City of Danville, Virginia

1.2.6 Purpose of Dam: The primary purpose is for hydroelectric power for the city of Danville. The reservoir provides the water storage to replenish the power pool in the Townes Dam Reservoir, which is located downstream of Talbott Dam.

1.2.7 Design and Construction History: The dam was designed in 1937 by Charles T. Main, Inc., Engineers and constructed in 1938 by Ligon and Ligon, contractors. With the exception of changing the regulating valves from one 3-foot diameter to two 3-foot diameter valves and using 6-inch bypasses in lieu of 4-inch, design drawings appear to reflect "as-built" conditions.

1.2.8 Normal Operational Procedures: Operation of the project is regulated. The 36-inch gated outlet is operated as needed to allow water to flow to the lower dam for hydro-electric power generation. At high pool levels, water is automatically passed over the spillway.

1.3 Pertinent Data

1.3.1 Drainage Areas: The dam controls a drainage area of 20.2 square miles.

1.3.2 Discharge at Dam Site:

Maximum known flood-10,300 c.f.s. in August 1940(estimated).

36-inch Outlet

Pool level at spillway crest without
flashboards - 376 c.f.s.

Spillway

Pool level at top of parapet wall with flashboards -
4000 c.f.s.

Pool level at top of parapet wall without flashboards -
7900 c.f.s.

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

Table 1.1 DAM AND RESERVOIR DATA

ITEM	Elevation Ft., msl	Area Acres	Acre feet	Watershed Inches	Length Miles
Top of Parapet	2543. 5	189	9600	8.9	2.75
Top of Walkway	2540.0	175	8900	8.3	2.65
Flood Level <u>1/</u>	2538.0	170	8700	8.1	2.6
Top of Flashboards	2535.0	157	8100	7.5	2.6
Spillway Crest	2532.0	150	7600	7.1	2.6
Normal Pool	2525-2531	-	-	-	-
Streambed at Center- line of Dam	2405	-	-	-	-

1/ Elevation noted on original drawings.

SECTION 2: ENGINEERING DATA

2.1 Design:

2.1.1 Design Drawings: The city of Danville has on file an incomplete set of original design drawings and specifications (See Appendix I) prepared by Charles T. Main, Inc., Boston, Massachusetts, dated 1937. These drawings contain information relating to core borings, foundation and grouting, boring and testing records of concrete, plan and section views of the dam and appurtinent structures, electrical wiring and mechanical piping.

2.1.2 Geologic Investigations: An extensive geologic and foundation investigation was conducted for the dam by the design engineers, Charles T. Main, Inc. The investigations were directed by the Consulting Geologist, Wilbur A. Nelson under contract to the design engineers. Geology reports are on file in the City of Danville Electric Dept. As part of the preliminary studies, eight separate sites were investigated. The investigations at each site included detailed geologic mapping and diamond core drilling. A total of 58 core borings were drilled with most concentrated at two of the more favorable sites. The favorable sites were determined on the basis of narrowness of the gorge and the geologic conditions at each site. The final site selected was between sites 21 and 22 where the gorge was narrowest. A site 550 feet upstream of 21 and 22 was the most geologically advantageous according to the consulting geologist. The selected site was, however, approved by the consulting geologist provided recommended foundation treatments were followed. In addition to the geologic mapping and drilling, the investigations also included several test pits, a detailed groundwater study on the left abutment of the selected site, and petrographic analysis of the foundation rocks.

2.1.2.1 Geologic Setting of the Dam Site: The dam is located in the Blue Ridge Physiographic Province of Virginia and is underlain by metamorphic rocks of the Precambrian Age Lynchburg Formation. These rocks include predominately quartz-mica gneiss with minor amounts of schist and quartzite. Massive greenstones of the Catoclin Formation occur 25 to 50 feet upstream of the dam site. The general strike of the schistosity of these rocks is from N45° to 60° with a dip to the southeast of 60° to 80°. The structural geology of the site is fairly complex. At least three periods of folding with associated jointing have occurred in the rocks of the area. Thrust and normal faulting are common in the Blue Ridge but neither occurs in the

immediate dam site. A near vertical, normal fault was uncovered in a cut for the service road but it strikes parallel to the ridge top west of the dam site and has no adverse affect at the dam location.

2.1.2.2 Foundation Condition and Treatment: The condition of the foundation rocks varied from abutment to abutment. Based on the core borings, overburden thicknesses ranged from 2 to 16 feet and depths to unweathered rock ranged from 6 to 54 feet. The deeper weathered zones occur high on the right abutment, where although the rock is deeply weathered, abutment conditions were determined to be excellent.

According to contract specifications inclosed in Appendix VI, excavations for the dam foundation were made to a sufficient depth to secure the foundation on sound bedrock free of open or soft seams. The excavations were made in horizontal steps at least 5 feet high as shown on Plate 4, Appendix I. After excavation of the foundation and before placement of the concrete structure, the base was thoroughly cleaned of all loose materials and a $\frac{1}{2}$ " thick layer of slush grout was applied to the cleaned rock surface to secure a good bond with the first lift of concrete. Steel dowel bars were also grouted into the bedrock before concrete placement.

Weathered zones were also encountered within the underlying competent bedrock along narrow planes parallel to the schistosity. These weathered planes in some areas extended below the river elevation and were fairly continuous. Since the northeast strike of the schistosity planes cut across the dam site at approximately 45° , leakage through the weathered planes was anticipated. To stop leakage under the dam and to provide a uniform and consolidated foundation, an extensive grouting program was completed before construction of the dam structure. The grouting was done in two phases. The first phase included drilling 35 grout holes to 30 feet with a diamond core drill along the dam perimeter. After examining the cores and the grout records for these holes, the second phase grout holes were laid out and drilled to a minimum depth of 30 feet with jackhammers. Approximately 460 second phase holes were drilled and grouted. The grouting procedure is outlined in the specifications in Appendix VI. Drawings showing the hole locations and grout records of both phases are shown on Plates 5, 6 and 10, Appendix I.

In addition to the normal investigation at the dam site, detailed studies of the narrow ridge comprising the left abutment were also performed. The studies were conducted to determine if leakage would occur through weathered schistosity planes oriented perpendicular to the narrow ridge. The ground water study and nine core borings drilled along the top of the ridge indicated zones of fractured rock and a ground water table that was lower than the proposed reservoir for a distance of 240 feet downstream of the left abutment. These conditions indicated leakage through the ridge was probable. Geologic

cross sections along the ridge are shown on Plate 8, Appendix I and the ground water study is discussed in the Geology report in Appendix IV. Grouting the 240-foot zone was recommended by the consulting geologist. No grout records were found among the drawings obtained from the City of Danville, therefore, it must be assumed the grouting was not accomplished.

2.1.3 Structural Analysis: Original design calculations were not found.

2.2 Construction: Construction drawings by Charles T. Main, Inc. detailing foundation grouting as well as a drawing showing concrete placement and results of concrete tests are available from the City of Danville. (See Plates 4, 5, and 6, Appendix I). No other construction records could be found.

2.3 Operations: The City of Danville employs a reservoir patrolman who lives within walking distance of the dam. His operational duties include reading the staff gage twice daily, operation of the outlet regulating valves as required, and other small maintenance tasks. Other operational personnel reside at Townes Dam and the Pinnacles Power Plant, both located downstream, but connected in operational procedure to Talbott Dam. The patrolman keeps a log of all operations.

2.4 Evaluation: The design drawings and geology report provide excellent information concerning the design and construction of the dam. However, without the original design analysis the available engineering data is insufficient to base an opinion on the safety of the dam.

SECTION 3 - VISUAL INSPECTION

3.1 General: Prior to the field inspection performed on 27 July 1978, the only known inspection was an underwater inspection performed by Logan Diving, Inc. A copy of their inspection report is inclosed in Appendix V. In general, their inspections indicated that there was an accumulation of logs and brush on the trash racks and an accumulation of silt along the face of the dam.

3.2 Findings:

3.2.1 Dam and Abutments: The results of the 27 July 1978 inspection are recorded in Appendix III. At the time of the inspection, the pool elevation was 2527.2, 7.8 feet below the elevation of the spillway. The outlet works were not releasing flow. Only minor seepage was noted on the face of the dam. There were some areas of calcium deposits showing areas of past seepage that are not currently active. A horizontal crack, approximately 1/2 inch wide and 30 feet long, is located at the bottom of the fourth lift from the top of the dam, beginning at the left abutment on the downstream face. The concrete above the crack is displaced approximately 1/8 inch. The crack is seeping at a rate estimated to be 1 GPM.

Verification was made as to foundation rock type and orientations of the schistosity. The abutments and downstream area were inspected for signs of foundation failure and leakage. A spring surrounded by a large wet area was noted along the narrow left abutment approximately 150 feet downstream and 50 feet below the top of the dam. The spring was flowing clear at approximately 10 gpm and, although no temperature reading was made, the water was very cool and therefore, not runoff. There was no mention of any springs along the left abutment in the geologist's report but the dam operator stated the spring had been there since he began work approximately 10 years ago. It is very probable that the spring developed as a result of leakage through weathered schistosity planes in the narrow ridge soon after filling the reservoir. Since grouting records were not found that would indicate the designers followed the recommendations of the consulting geologist, it is assumed the grouting was not done and that the spring is leakage that has occurred as a result. Several sections of 4" clay pipe were found below the spring and were probably part of a drain system placed to control erosion. The system was not operating and a large area around and below the spring was saturated. The presence of the old drain system may further substantiate the development of the spring since construction of the dam. Besides the spring, no other wet areas or zones of seepage were noted along the abutments or downstream area.

3.2.2 Reservoir Area: Conversations with representatives of the city of Danville indicate that divers reported moderate siltation and accumulation of trash near the dam face. The side slopes of the reservoir are very steep.

3.2.3 Electrical: Electrical power to the dam is supplied by a 2,300 volt, three-phase transmission line from the Pinnacles Power plant. An oil filled distribution type transformer in the storage room provides power to the upper portion of the dam. Two oil filled distribution type transformers in the valve house provide power for operation of the valve. A backup power source is not available. Location of the transformers violates the National Electrical Code. Emergency lighting is not available in the valve house. Inspectors noted that the transformers have some oil leakage around bushings or lid.

3.2.4 Downstream Areas: A U.S.G.S. quad sheet was used as a guide to assess the areas which might be affected by a dam failure. Townes Dam is located downstream of Talbott Dam and 25 dwellings are located immediately downstream of Townes Dam. The dam is located in a gorge with steep side slopes, approximating 50 degrees.

3.3 Evaluation:

3.3.1 Dam and Abutment: The dam and abutments were in generally good condition.

3.3.2 Downstream Area: The area downstream of the dam is clear of any structure with the exception of the Townes Dam. A failure of the Talbott Dam would probably cause a failure in the Townes Dam. A failure of the Townes Dam could result in the destruction of many dwellings.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures and Maintenance: The maximum storage pool is elevation 2535 which is the top of the flashboards or three feet above the spillway crest. The reservoir provides the water storage to replenish the Townes Dam reservoir (located $5\frac{1}{2}$ miles downstream) which provides water for hydroelectric power generation. Water flows from the reservoir through a 36-inch diameter steel conduit that discharges into the stilling basin of the dam. Under normal streamflow conditions, the reservoir pool ranges between elevations 2525 and 2531. Excess streamflows are automatically passed over the spillway. Flows through the outlet works are controlled either electrically or manually by the patrolman stationed at Talbott Dam. A private telephone system provides communications between Talbott Dam, the power plant, Townes Dam, and the patrolman's house.

4.2 Maintenance of Dam: Routine maintenance of the dam is generally not required due to the type of structure. Debris and logs are routinely removed from the reservoir by the partolman permanently stationed at Talbott dam. His primary duties are to maintain, operate and inspect the dam and operating facilities.

4.3 Maintenance of Operating Facilities: Maintenance schedules are maintained for Talbott dam requiring lubrication of valve fittings and checks of master controls weekly as well as numerous minor maintenance tasks.

In 1971 the city replaced, with new counterparts, the downstream gate valves and all of the 90-degree elbow (extra strong) pipe fittings in both 6-inch bypasses that serve the 36-inch valves.

In 1975 the 36-inch (Service) butterfly valve was removed and transported to the Chapman Division of the Crane Company, Indian Orchard, Massachusetts, for complete overhaul. The removal, transportation, and reinstallation was accomplished by city personnel.

In April 1978 the stainless steel hoist rope and the single block at the stop gate were replaced with new counterparts. The above new material was furnished by the city and was installed by Logan Diving, Inc. under an inspection contract. (See Appendix V). The main requirements of the contract involved the underwater inspection of the fixed trash rack; the inspection and operation of the stop gate and its hoist winch, both 36-inch butterfly valves, all gate valves in both 6-inch bypasses, and in the 6-inch drain line; also the inspection of the twenty-six nozzles in the discharge ring at the stilling basin. The operating phase of the inspection was accomplished by city personnel in the presence of Logan Diving, Inc. personnel. The inspection revealed that the trash rack was obstructed for approximately two-thirds of its height. The obstruction consisted of a composite mixture of silt,

leaves, and waterlogged tree trunks, limbs, and branches. The condition and the operation was found to be satisfactory for the stop gate and its hoist winch, the two 36-inch butterfly valves, and the four gate valves in the two 6-inch bypasses. The gate valve in the 6-inch drain line was found to be defective and was replaced with a new counterpart. It was found that two of the twenty-six nozzles in the discharge ring were clogged.

4.4 Warning System: No formal warning systems exist.

4.5 Evaluation: Operation and maintenance of this dam by the City of Danville Electric Department appear good. Additions to present procedure are indicated in Section 7, Assessment/Remedial Measures.

SECTION 5: HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: There is no original hydraulic or hydrologic design data available for the Talbott Dam.

5.2 Hydrologic Records: Records of reservoir pool elevations, air temperature, and precipitation are maintained at the Pinnacles Hydro-Electric Power Plant. The reservoir content at the end of each month are published by the U. S. Geological Survey in "Water Resources for Data for Virginia". The U. S. Geological Survey has maintained flow records approximately 36½ miles downstream on the Dan River near Francisco, N.C. (drainage area 124 square miles) since August 1924.

5.3 Flood Experience: The maximum pool level reported for the Talbott Dam occurred 14 August 1940 at an elevation of 2546 ft, MSL, or 2.5 feet above the top of the parapet wall. This is the only incident in which the parapet wall has been overtopped. The reservoir pool reached elevation 2538.5 ft, MSL during Tropical Storm Agnes in June 1972.

5.4 Flood Potential: The Probable Maximum Flood (PMF), 1/2 PMF, and one percent flood hydrographs were developed and routed through the reservoir by use of the HEC-1DB computer program (Reference 1, Appendix VII) and appropriate unit hydrograph, precipitation, and storage-outflow data. Clark's Tc and R coefficients for the local drainage area were estimated from basin characteristics and previously derived coefficients at the Francisco gage. The rainfall applied to the developed unit hydrograph was obtained from U. S. Weather Bureau Publications (References 2 and 3, Appendix VII). Losses were estimated at an initial loss of 0.80 inch and a constant loss thereafter of 0.05 inch/hour.

5.5 Reservoir Regulation: Releases from Talbott are regulated to supply the downstream Townes Reservoir as needed. The Townes reservoir is used for the generation of electricity for the city of Danville by the Pinnacles Hydro-Electric Power Plant. Under normal streamflow conditions, the reservoir pool ranges between elevations 2525 feet and 2531 feet. Excess streamflows are automatically passed over the spillway or flashboards.

The storage curve supplied by the owners was extended above the top of the flashboards by use of U. S. Geological Survey Quadrangle Maps. In the August 1940 flood, the flashboards did not blow out as designed. For this reason, two rating curves were developed for the spillway. One rating assumed the flashboards in place while the other assumed full conveyance for the ogee shaped spillway. A rating curve was also developed for the top of the parapet wall. All ratings assumed that the parapet wall would withstand the pressures associated with overtopping. In routing hydrographs through the reservoir, it was assumed that the initial pool level was at the top of the flashboards for the condition with flashboards in place and at the spillway crest for the condition without flashboards.

5.6 Overtopping Potential: The probable rise of the reservoir for the two conditions with and without flashboards and other pertinent information on reservoir performance is shown in the following table:

TABLE 5.1 RESERVOIR PERFORMANCE
FLOOD

	One Percent ^{1/}	1940	1/2 PMF	PMF ^{2/}
Peak Inflow, cfs	7,890	-	17,310	34,620
WITH FLASHBOARDS				
Peak Outflow, cfs	6,340	10,300	16,900	34,280
Peak Elevation, ft, msl	2544.6	2,546	2,547.5	2,550.7
Spillway, 2535.0 ft, msl				
Depth of Flow, ft ^{3/}	3.5	3.5	3.5	3.5
Avg. Velocity, fps	15.4	16.9	19.4	23.4
Top of Parapet Wall, 2543.5 ft, msl				
Depth of Flow, ft ^{4/}	1.1	2.5	4.0	7.2
Duration, hrs	5.0	-	11.5	16.0
Avg. Velocity, fps	3.5	5.2	6.6	8.9
Tailwater Elevation, ft, msl	2419.6	-	2433.4	2448.2
WITHOUT FLASHBOARDS				
Peak Outflow, cfs	6,150	-	16,730	34,290
Peak Elevation, ft, msl	2540.2	-	2,546.3	2,549.9
Spillway, 2532.0 ft, msl				
Depth of Flow, ft ^{3/}	6.5	-	6.5	6.5
Avg. Velocity, fps	11.8	-	19.2	21.2
Top of Parapet Wall, 2543.5 ft, msl				
Depth of Flow, ft ^{4/}	-	-	2.8	6.4
Duration, hrs	-	-	7.0	12.0
Avg. Velocity	-	-	5.5	8.3
Tailwater Elevation, ft, msl	2,419.2	-	2,433.3	2,448.2

^{1/} The One Percent Exceedence Frequency Flood has one chance in 100 of being exceeded in any given year.

^{2/} The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

^{3/} Depth of flow on spillway is limited due to walkway.

^{4/} Difference between the maximum reservoir elevation and the top of the parapet wall.

5.7 Reservoir Emptying Potential: A 36-inch gated outlet at elevation 2411.5 is available for dewatering the reservoir. There are no methods available for lowering the reservoir pool below this elevation. With the reservoir pool at the top of the flashboards, it would take approximately 16 days to lower the pool to elevation 2411.5 ft, msl.

5.8 Evaluation: The probability of the flashboards being blown out during flood conditions is uncertain; therefore, prime consideration was given to conditions of overtopping with the flashboards in place. Under this condition, the parapet wall was overtopped by 2.5 feet in the August 1940 flood. With the flashboards in place, the parapet wall would be overtopped by 1.1 feet in the One Percent Flood, 4.0 feet in the 1/2 PMF, and 7.2 in the PMF for durations of 5.0, 11.5, and 16.0 hours respectively. The parapet wall would be just overtopped by 20% of the PMF. Conditions would be slightly less severe if the flashboards were not in place in that 35% of the PMF would just overtop the parapet wall with overtopping by other floods shown in Table 5.1. The spillway is considered seriously inadequate since it will not pass one half of the PMF. If structural calculations indicate the dam is stable the spillway should be considered inadequate but not seriously inadequate.

Conclusions pertain to present day conditions and the effect of future development on the hydrology has not been considered.

SECTION 6 - STRUCTURAL STABILITY

6.1 Dam Foundation: Talbott Dam is founded on competent metamorphic bedrock along the river valley and both abutments. Foundation treatments including contact and consolidation grouting were accomplished under the entire dam structure. Foundation drains were determined to be unnecessary and therefore not constructed. For detailed description of site geology and foundation conditions and treatment, see Section 2, Engineering Data.

6.2 Stability: Structural calculations are not available and therefore, cannot be checked. (Analysis of stability and state of stress is beyond the scope of this report.)

6.3 Evaluation:

6.3.1 Structural: Visual observations do not reveal any problems which indicate instability. The one half inch horizontal crack at the left abutment (30 feet long and approximately 40 feet below the walkway at the top of the dam, see photos 5 and 6) does not appear to warrant serious concern at this time, however, this condition should be checked when a structural analysis is performed. Structural analysis under critical loading conditions should be completed within 6 months.

6.3.2 Foundation: Dams are usually evaluated on the basis of potential settlement, sliding and seepage. Settlement of the dam is not a problem because the foundation is competent bedrock. Abutment stability against sliding is critical in concrete arch dams since the abutments are absorbing most of the loads. In hard competent metamorphic rocks like the gneiss comprising both abutments, sliding is only critical where low angle weathered schistosity planes or shear zones occur dipping in either an upstream or downstream direction away from the abutments. No shear zones were mentioned in the geology report and, although weathered schistosity planes do occur dipping downstream, the magnitude of the dip is too great (60° - 80°) for sliding to occur. The potential for leakage did exist within the foundation rock, therefore, a grouting program was completed under the entire dam structure.

The program was fairly extensive and based on the visual inspection, quite effective as very little seepage was noted along the dam-abutment contact. Seepage through the narrow ridge comprising the left abutment was apparent in the form of a spring flowing 10 gpm. As previously stated, the development of the spring could have been the result of deleting the recommended grouting along ridge. Regardless, the spring being 150 feet downstream is presently not affecting the integrity of the dam. Since the water was clear, it is also apparent

that no piping or enlarging of the permeable zones in the bedrock is taking place. Since the grouting of the ridge was apparently deleted, additional leakage may develop through the abutment in the future. Periodic inspection of the abutment to locate additional leakage should be made. In addition, some simple measuring device should be installed at the existing spring to monitor any changes in flow associated with changes in reservoir levels. In the future, should increased flow of the spring and additional springs or seepage areas develop, then some remedial treatment of the abutment may be required to insure the safety of the dam.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS/ REMEDIAL MEASURES

7.1 Assessment:

7.1.1 Safety: The Talbott Dam, as observed 27 July 1978, appears sound without indication of structural instability or unsafe operation. The spillway will pass only 20% of the PMF without overtopping the parapet (flashboards in) and it will not pass the 100 year flood without exceeding the flood level, elevation 2538, indicated on the drawings. This is cause for concern since no stability analysis is available and the dam may have only been structurally designed for the flood level shown on the drawings. With the flashboards removed, the spillway will pass 35% of the PMF. The spillway is therefore adjudged as seriously inadequate and the dam is assessed as unsafe, non-emergency.

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean, however, that based on an initial screening and preliminary computations, there appears to be a serious deficiency in spillway capacity so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream from the dam.

7.1.2 Adequacy of Information: With the exception of structural analysis, information on the dam is excellent.

7.1.3 Urgency: Preparation of structural analysis should begin immediately.

7.1.4 Necessity of Phase II: A Phase II inspection is not considered necessary.

7.2 Recommendations/Remedial Measures:

7.2.1 Structural Calculations: Structural analysis of the dam, including a check of the horizontal crack at the left abutment noted in paragraph 3.2.1, should be started immediately and completed within 6 months.

7.2.2 In accordance with paragraph 7.1.1, it is recommended that within two months from the date of notification to the Governor of the Commonwealth of Virginia, the owner engage the services of a professional consultant to determine by more sophisticated methods and procedures the adequacy of the spillway. Even though the seriously inadequate spillway would produce a dam failure primarily from hydrologic reasons, remedial measures in structural or geotechnical areas may be needed to remove the dam from an unsafe classification. Within 6 months of the date of notification to the governor, the professional consultant's report of appropriate remedial mitigating measures should have been completed and the owner should have an agreement with the Commonwealth of Virginia to a reasonable time frame in which all remedial measures will be complete. In the interim, a detailed emergency operation plan and warning system should be promptly developed. It must be considered that the dam is threatened whenever the reservoir level exceed elevation 2538 until analysis shows otherwise. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

7.2.3 Flashboards: Flashboards should be removed until the stability of the dam is verified. (This reduces the probability of activating the warning system to approximately one in one hundred for any given year.)

7.2.4 Stop Gate Inspection: It is recommended, as outlined in the Logan Diving, Inc. report, that the city perform a semi-annual operational inspection of the intake stop gate and its hoist winch, the two 36-inch butterfly valves, the four gate valves in the two 6-inch bypasses, and the gate valve in the 6-inch drain line. The inspection should be performed in the presence of the superintendent of the Pinnacles Hydro Station. The inspection report should include the name of the dam, the date, the names of the inspectors, the weather conditions, the reservoir pool elevation, and other pertinent information. The results of the inspection should be described with special emphasis being placed on any deficiency that is revealed.

7.2.5 Underwater Inspections: Although no apparent operational problems have been experienced as a result of the excessive silt deposits on the trash rack, and from the clogged nozzles in the discharge ring, it is recommended that annual underwater inspections be made at these two areas to forestall serious operational problems in the future. Appropriate reports should be made of such inspections.

7.2.6 Mechanical Equipment: In connection with the maintenance of the manually-operated and the motorized equipment at this dam, there was no apparent evidence of neglect. It is believed, however, that the city should contact each manufacturer involved to determine if any modifications should be incorporated in so far as the lubrication types and application schedules are concerned.

7.2.7 Electrical Relocation: The transformers, fuse cutouts, and surge arresters should be relocated outside of the storage room and valve house.

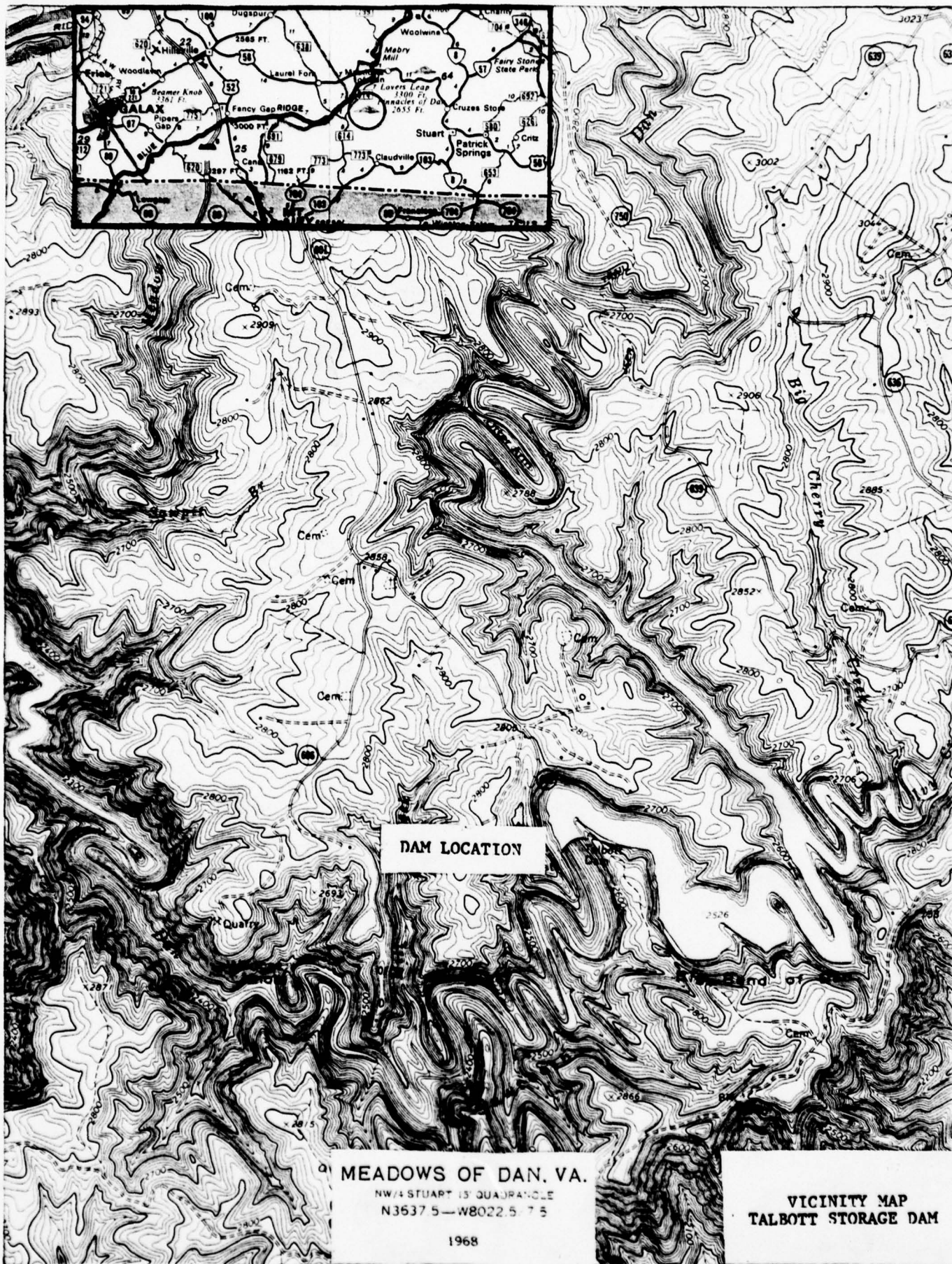
7.2.8 Transformer Oil Leakage: Transformer oil leakage should be repaired and the dielectric strength of the oil should be tested.

7.2.9 Emergency Lighting: Battery operated emergency lighting should be provided in the valvehouse.

7.2.10 Left Abutment Inspection: The left abutment should be inspected semi-annually to locate any additional leakage.

7.2.11 Seepage Monitoring: A simple monitoring device should be installed at the existing spring on the left abutment to monitor any changes in flow associated with changes in reservoir level. Flow should be measured every three months and additionally during periods of high reservoir levels. Should increased flow of the spring and additional springs or seepage areas develop, then some remedial treatment of the abutment may be required to insure the safety of the dam.

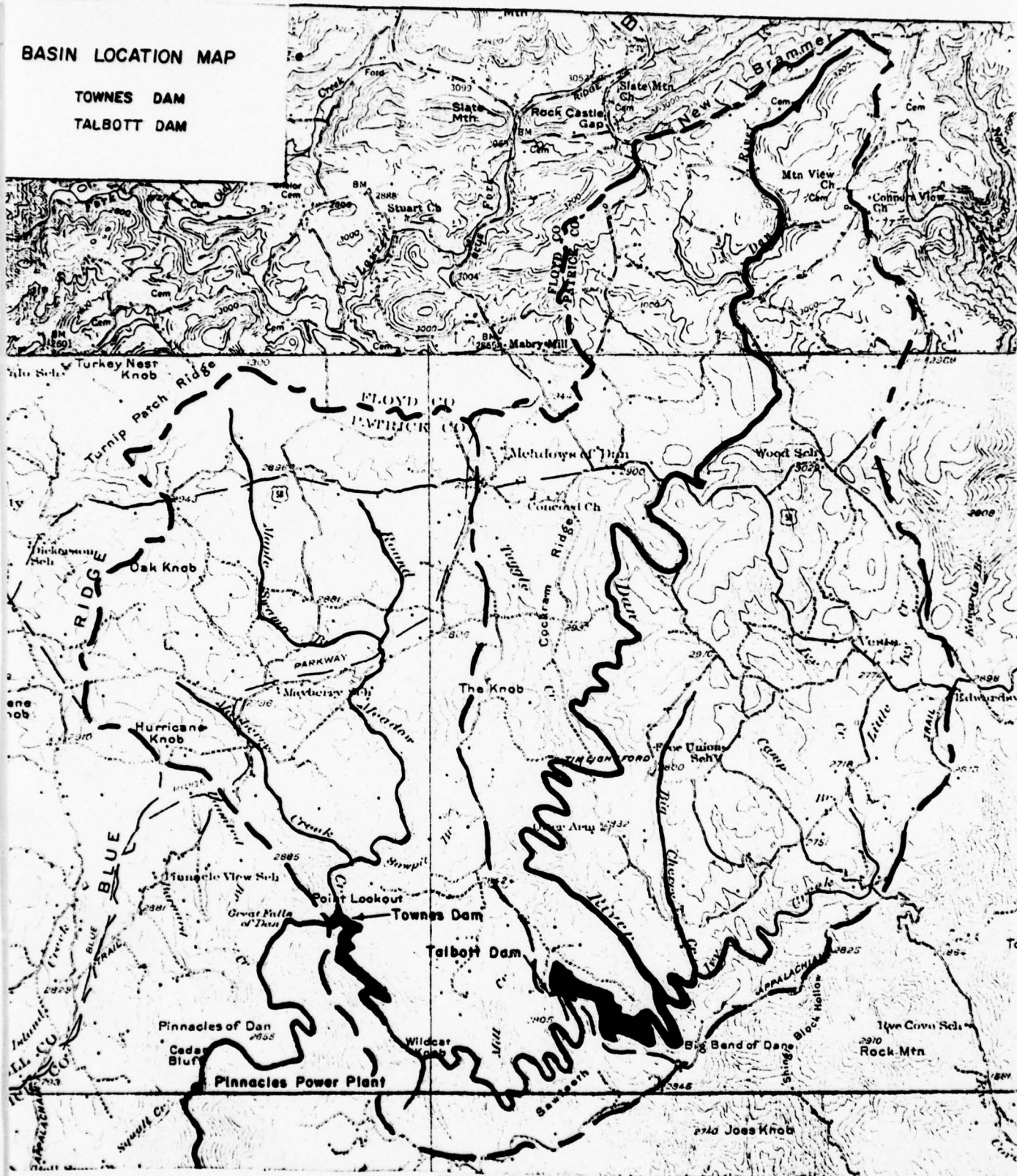
APPENDIX I
MAPS AND DRAWINGS



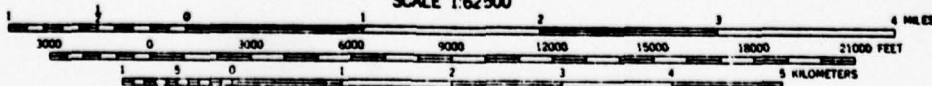
BASIN LOCATION MAP

TOWNES DAM

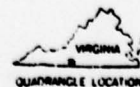
TALBOTT DAM

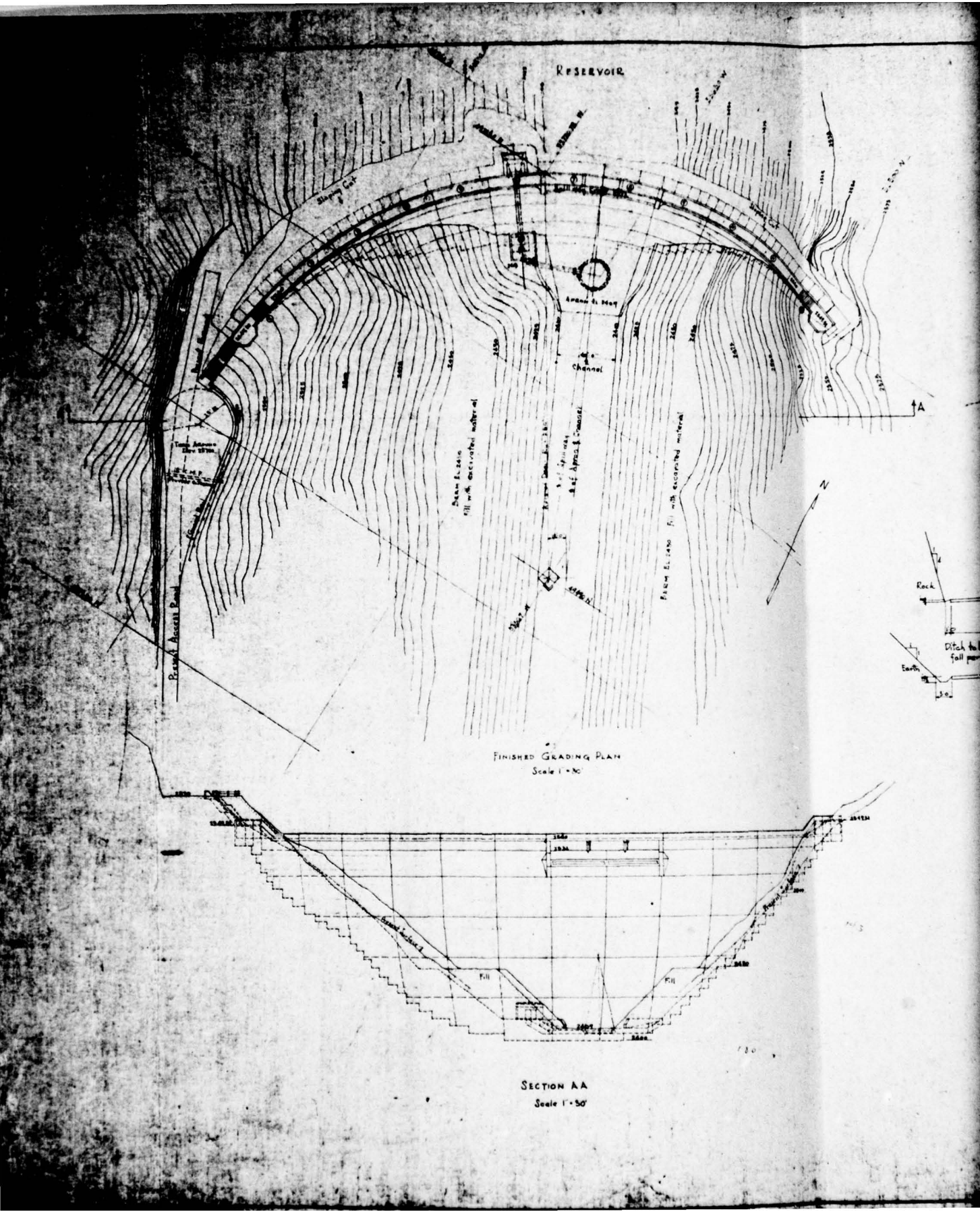


SCALE 1:62500



CONTOUR INTERVAL 40 FEET
DATUM IS MEAN SEA LEVEL





RESERVOIR

Approx. El. 2400

Channel

Fill with compacted material

Approx. El. 2400

Fill with compacted material

↑ A

N

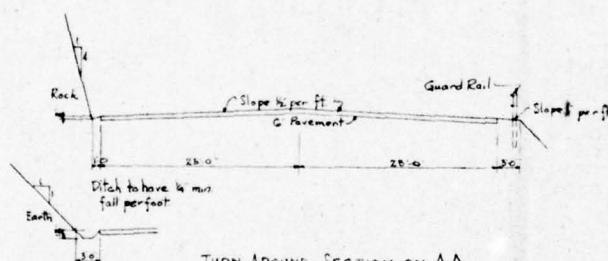
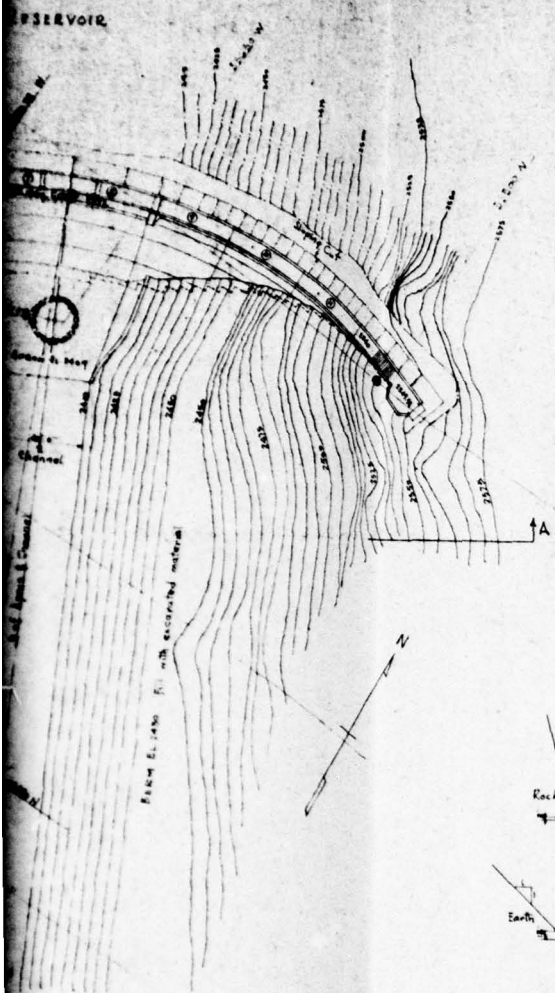
Rock

Ditch to fall pump

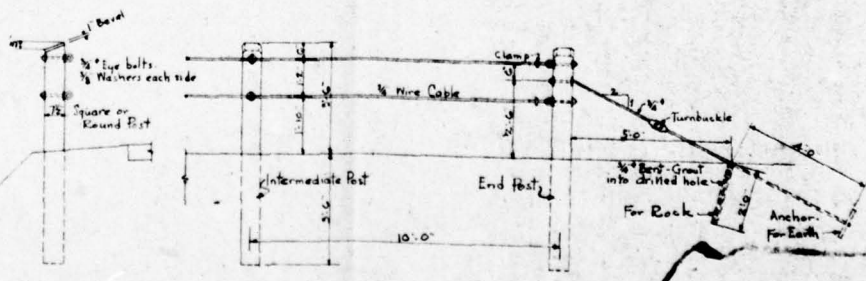
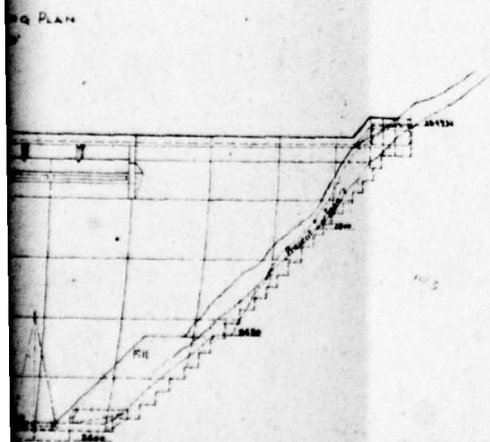
Earth

FINISHED GRADING PLAN
Scale 1" = 50'

SECTION AA
Scale 1" = 50'



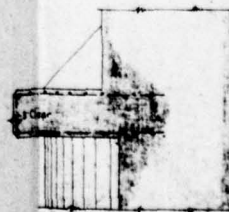
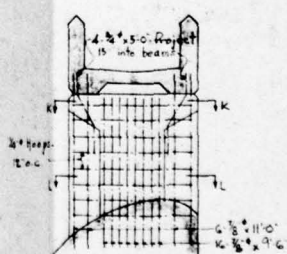
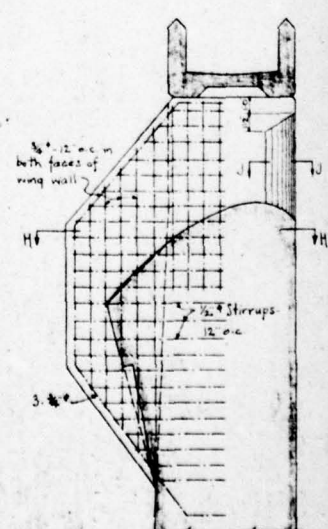
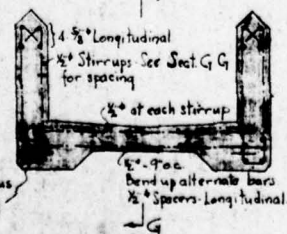
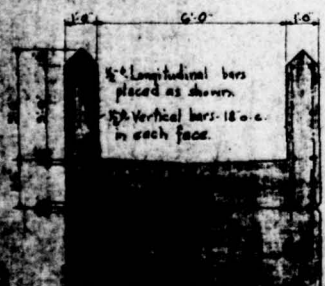
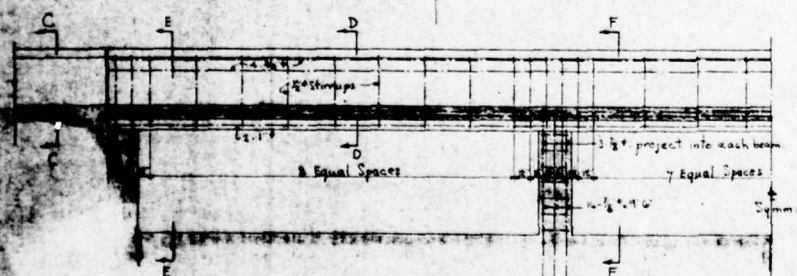
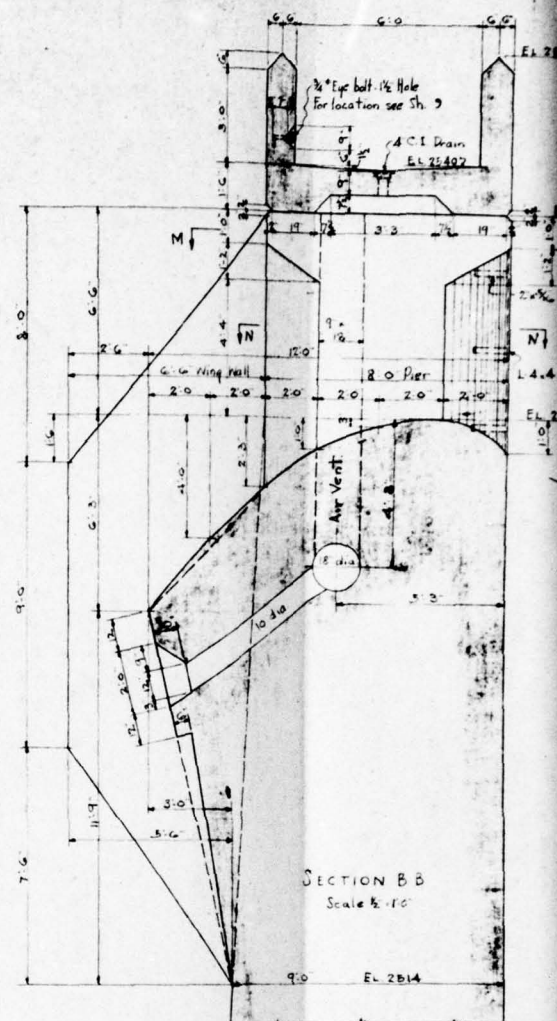
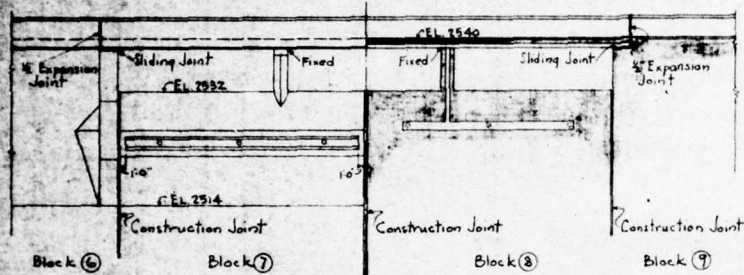
TURN AROUND SECTION ON AA
Scale 1/2"=10'

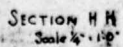
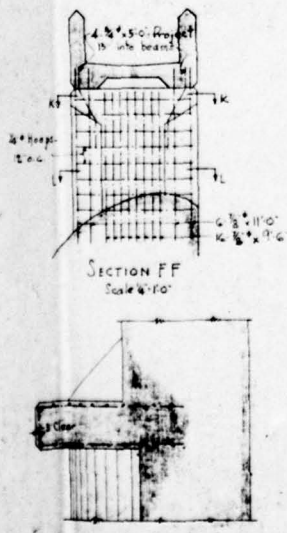
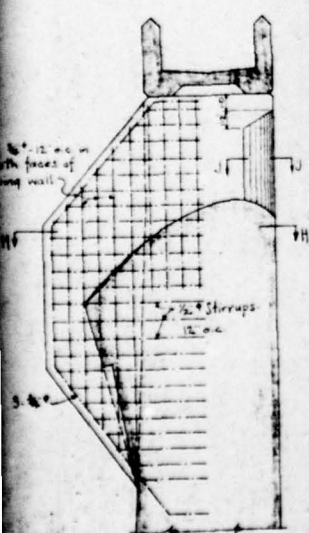
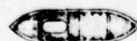
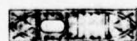
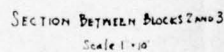
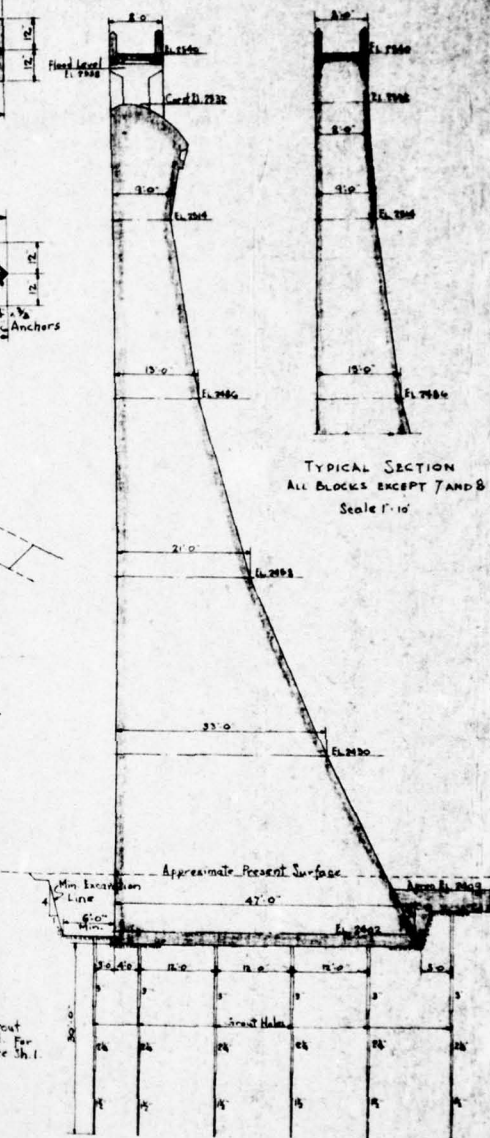
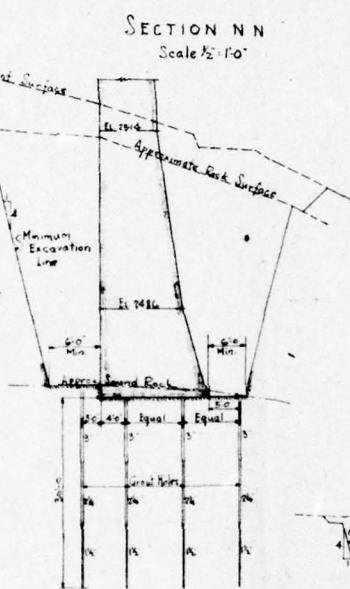
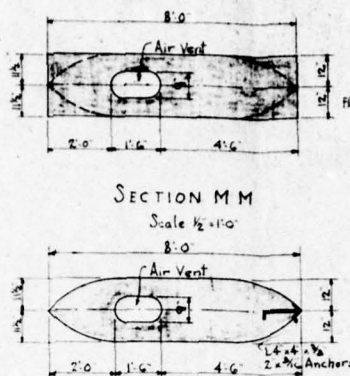
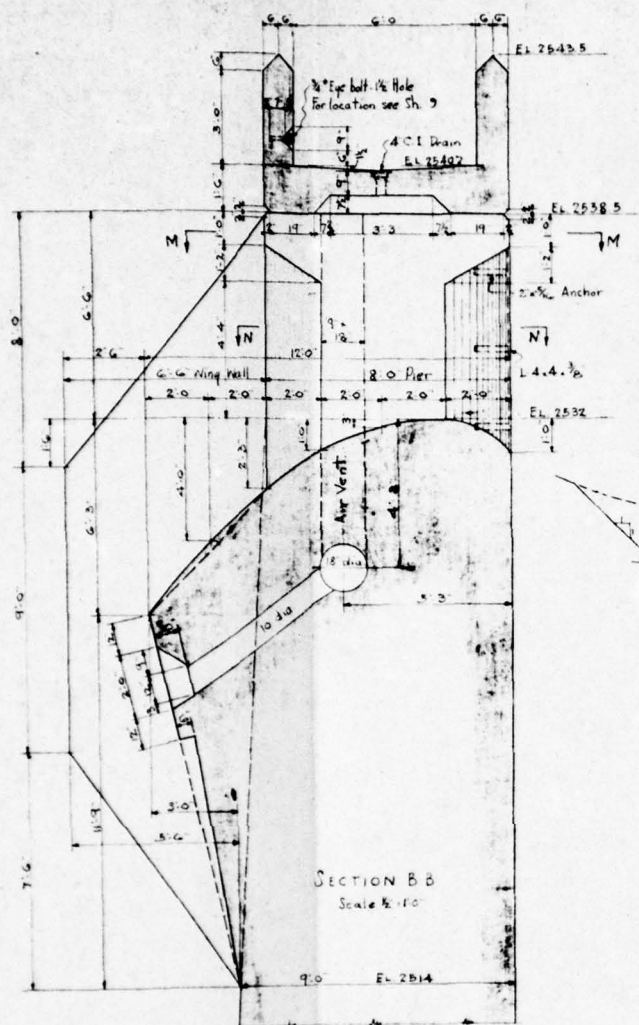


GUARD RAIL DETAILS
Scale 1/2"=10'

PLATE I

PWA DOCKET No. 1151-P2
SHEET 7 OF 15 SHEETS
PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
BIG BEND DAM
GRADING PLAN AND SECTION
CHAS. T. MAIN, INC., ENGINEERS
201 BAYVIEW ST. DANVILLE, VA.
REVISIONS





Minimum size of grout holes as indicated. For location of holes see Sh. 1

Notes
Above Elev. 7514, Steel forms shall be used to form the Spillway and the walkway piers. Below Elev. 7514, wooden forms may be used.
Sliding joints in walkway to be provided by placing 3 ply of 15lb Asphalt Saturated felt trimmed neat to whole size of the joint area.
Galvanized steel forms for all air vents to be left in place.

PWA DOCKET No. 1151-PR

SHEET 3 OF 13 SHEETS

PINNACLES HYDRO-ELECTRIC PROJECT

CITY OF DANVILLE ELECTRIC DEPARTMENT

DANVILLE VIRGINIA
BIG BEND DAM

BLOCKS 6-7-849

SPILLWAY DETAILS

CHART I M

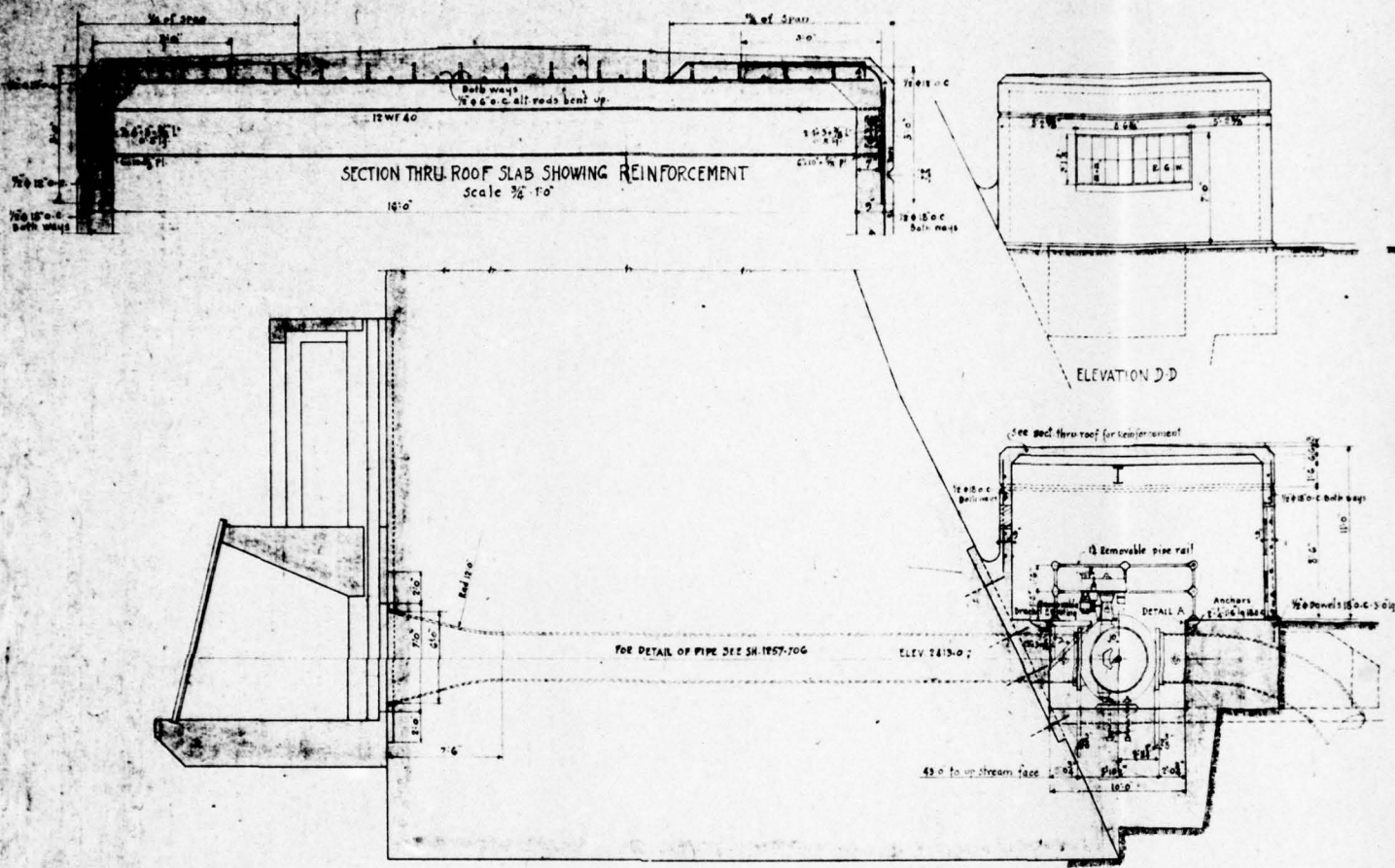
FOUNDATION GROUTING

ENGINEER

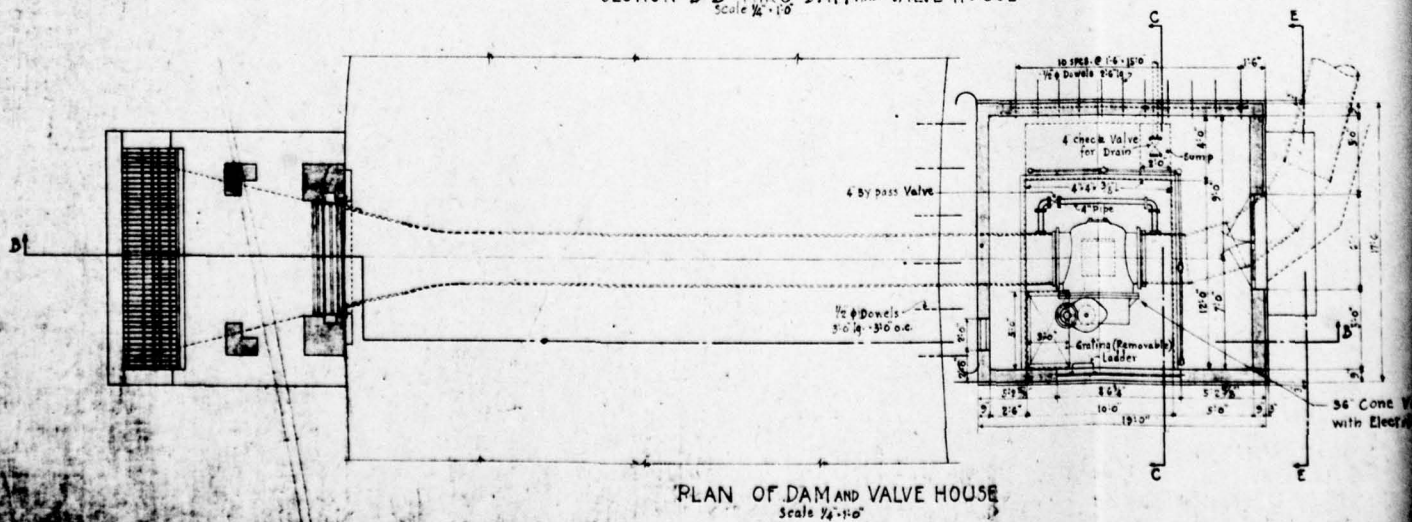
CHAS. T. MAIN, INC. ENGINEERS
301 DEVEREUX ST. BOSTON, MASS. U.S.A.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72	73	74	75	76	77	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95	96	97	98	99	100	101	102	103	104	105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128	129	130	131	132	133	134	135	136	137	138	139	140	141	142	143	144	145	146	147	148	149	150	151	152	153	154	155	156	157	158	159	160	161	162	163	164	165	166	167	168	169	170	171	172	173	174	175	176	177	178	179	180	181	182	183	184	185	186	187	188	189	190	191	192	193	194	195	196	197	198	199	200	201	202	203	204	205	206	207	208	209	210	211	212	213	214	215	216	217	218	219	220	221	222	223	224	225	226	227	228	229	230	231	232	233	234	235	236	237	238	239	240	241	242	243	244	245	246	247	248	249	250	251	252	253	254	255	256	257	258	259	260	261	262	263	264	265	266	267	268	269	270	271	272	273	274	275	276	277	278	279	280	281	282	283	284	285	286	287	288	289	290	291	292	293	294	295	296	297	298	299	300	301	302	303	304	305	306	307	308	309	310	311	312	313	314	315	316	317	318	319	320	321	322	323	324	325	326	327	328	329	330	331	332	333	334	335	336	337	338	339	340	341	342	343	344	345	346	347	348	349	350	351	352	353	354	355	356	357	358	359	360	361	362	363	364	365	366	367	368	369	370	371	372	373	374	375	376	377	378	379	380	381	382	383	384	385	386	387	388	389	390	391	392	393	394	395	396	397	398	399	400	401	402	403	404	405	406	407	408	409	410	411	412	413	414	415	416	417	418	419	420	421	422	423	424	425	426	427	428	429	430	431	432	433	434	435	436	437	438	439	440	441	442	443	444	445	446	447	448	449	450	451	452	453	454	455	456	457	458	459	460	461	462	463	464	465	466
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1990年12月31日

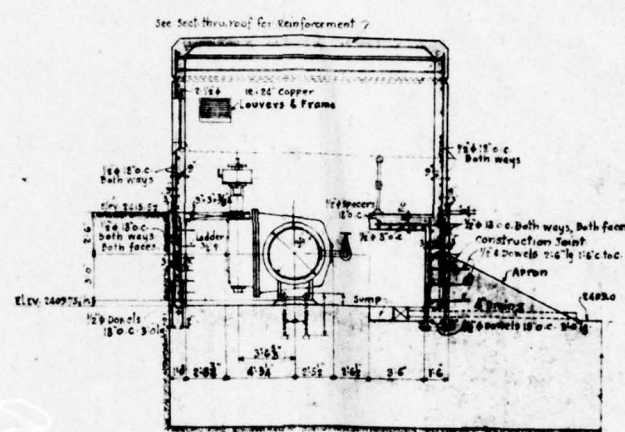
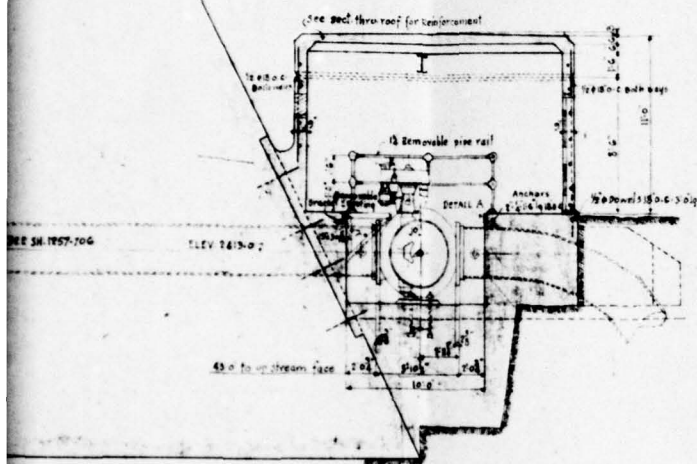
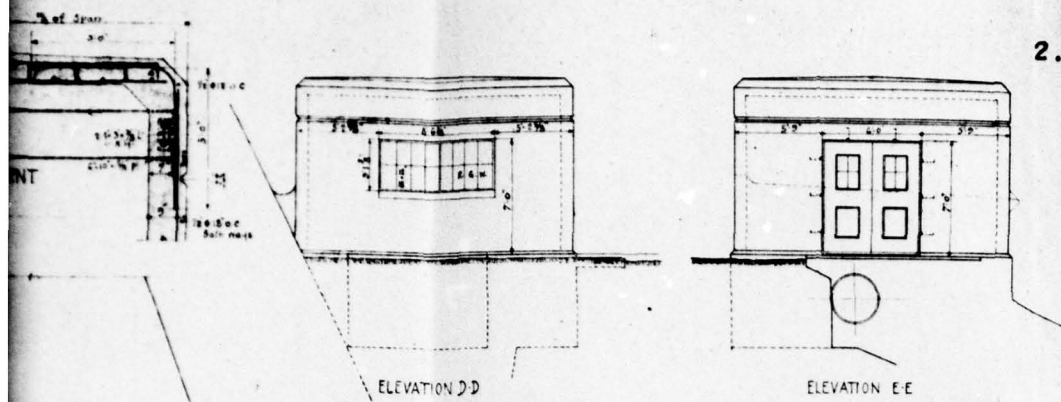


SECTION B-B THRU DAM AND VALVE HOUSE
Scale $\frac{1}{4}" = 1'-0"$



INSPECTOR'S NOTES

1. 6" Valves & Piping were found in lieu of 4" shown.
2. Two 36" inch Tandem Butterfly Valves were found in lieu of one 36" inch valve as shown.



PLAN OF DAM AND VALVE HOUSE
Scale 1/4" = 1'-0"

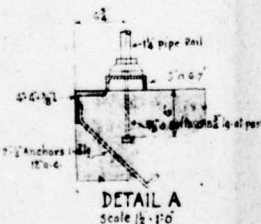
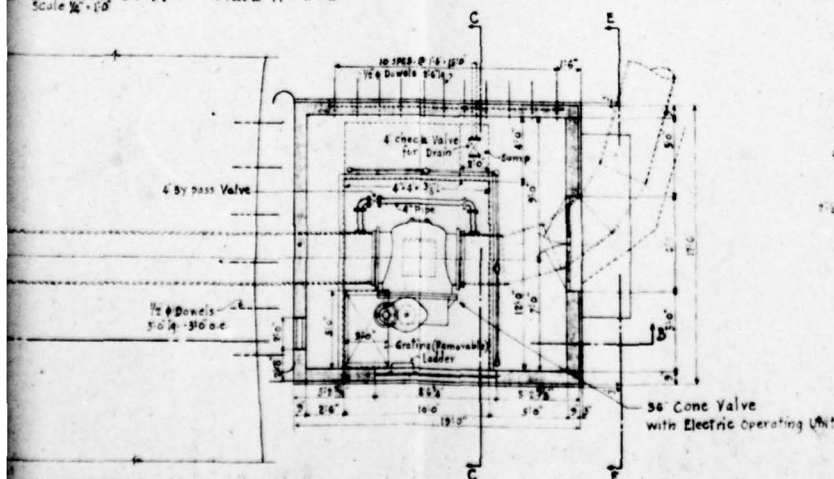


PLATE 3

P.W.A. DOCKET NO. 151 P.R.
SHEET 5 OF 13 SHEETS

PINNACLES HYDRO-ELECTRIC PROJECT.

CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE, VIRGINIA
TALBOTT DAM

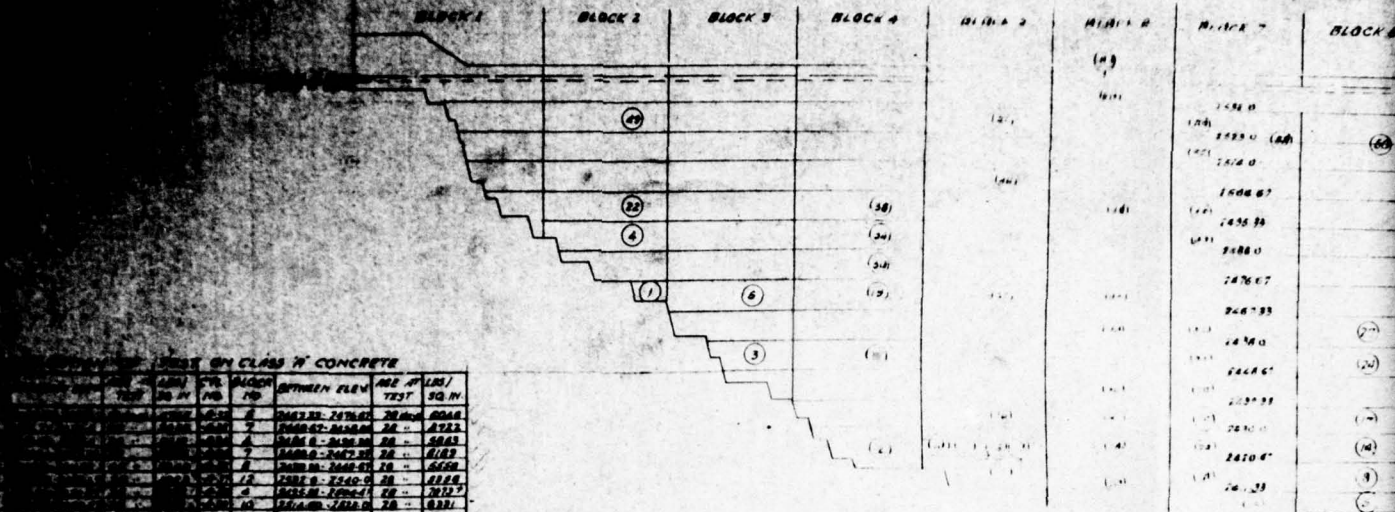
BLOCK 6 VALVE HOUSE AND DETAILS

CHAS. T. MAIN, INC., ENGINEERS
201 BROADWAY, N.Y.C.

DESIGNED BY CHAS. T. MAIN, INC.

CHECKED BY CHAS. T. MAIN, INC.

APPROVED BY CHAS. T. MAIN, INC.



TEST DATA CLASS A CONCRETE

TEST NO.	AGE AT TEST (DAYS)	TEST RESULT (PSI)	TEST RESULT (MPA)
1	28	4000	27.6
2	28	4000	27.6
3	28	4000	27.6
4	28	4000	27.6
5	28	4000	27.6
6	28	4000	27.6
7	28	4000	27.6
8	28	4000	27.6
9	28	4000	27.6
10	28	4000	27.6
11	28	4000	27.6
12	28	4000	27.6
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15	28	4000	27.6
16	28	4000	27.6
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36	28	4000	27.6
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38	28	4000	27.6
39	28	4000	27.6
40	28	4000	27.6
41	28	4000	27.6
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43	28	4000	27.6
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45	28	4000	27.6
46	28	4000	27.6
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71	28	4000	27.6
72	28	4000	27.6
73	28	4000	27.6
74	28	4000	27.6
75	28	4000	27.6
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78	28	4000	27.6
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82	28	4000	27.6
83	28	4000	27.6
84	28	4000	27.6
85	28	4000	27.6
86	28	4000	27.6
87	28	4000	27.6
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89	28	4000	27.6
90	28	4000	27.6
91	28	4000	27.6
92	28	4000	27.6
93	28	4000	27.6
94	28	4000	27.6
95	28	4000	27.6
96	28	4000	27.6
97	28	4000	27.6
98	28	4000	27.6
99	28	4000	27.6
100	28	4000	27.6

TEST DATA CLASS B CONCRETE

TEST NO.	AGE AT TEST (DAYS)	TEST RESULT (PSI)	TEST RESULT (MPA)
1	28	4000	27.6
2	28	4000	27.6
3	28	4000	27.6
4	28	4000	27.6
5	28	4000	27.6
6	28	4000	27.6
7	28	4000	27.6
8	28	4000	27.6
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17	28	4000	27.6
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23	28	4000	27.6
24	28	4000	27.6
25	28	4000	27.6
26	28	4000	27.6
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28	28	4000	27.6
29	28	4000	27.6
30	28	4000	27.6
31	28	4000	27.6
32	28	4000	27.6
33	28	4000	27.6
34	28	4000	27.6
35	28	4000	27.6
36	28	4000	27.6
37	28	4000	27.6
38	28	4000	27.6
39	28	4000	27.6
40	28	4000	27.6
41	28	4000	27.6
42	28	4000	27.6
43	28	4000	27.6
44	28	4000	27.6
45	28	4000	27.6
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91	28	4000	27.6
92	28	4000	27.6
93	28	4000	27.6
94	28	4000	27.6
95	28	4000	27.6
96	28	4000	27.6
97	28	4000	27.6
98	28	4000	27.6
99	28	4000	27.6
100	28	4000	27.6

TEST CYLINDERS MADE FOR EACH 400 CY. OF CLASS A CONCRETE
 101 TEST CYLINDERS MADE FOR
 AVERAGE COMPRESSIVE STRENGTH AT 28 DAYS 4221 PSI 29.3 MPA

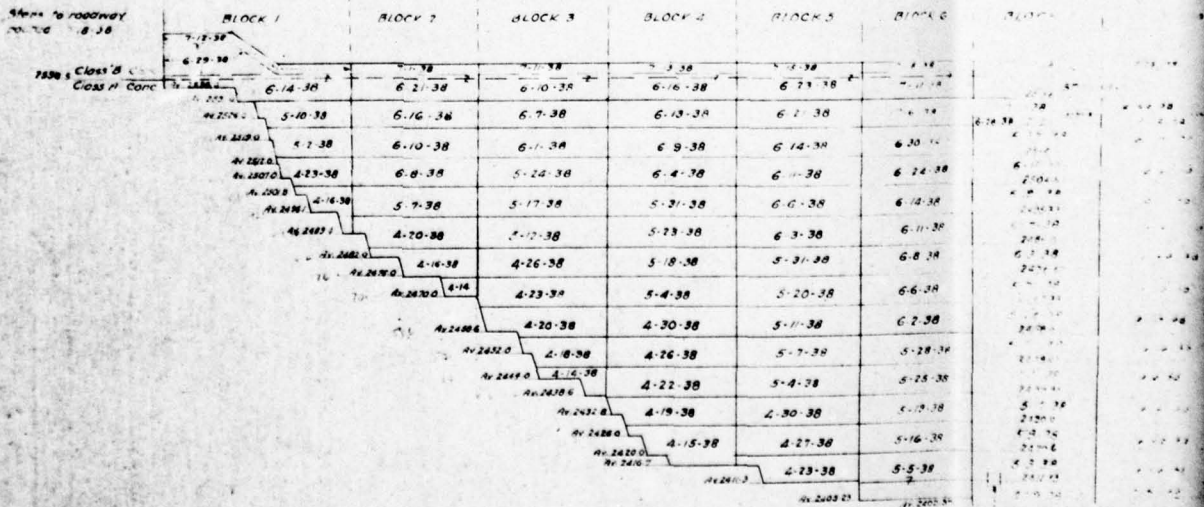


CHART SHOWING POURING DATES FOR EACH 400 CY. OF CLASS A CONCRETE
 FIRST CONCRETE POURING DATE 1964

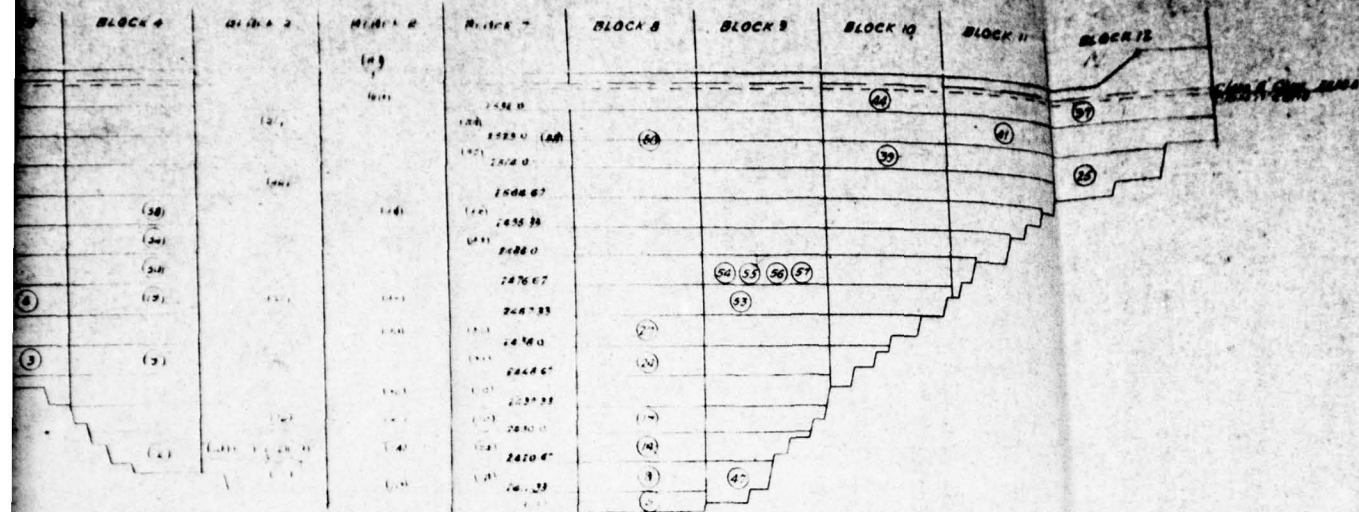


CHART SHOWING POURING DATES AND RISE OF CONCRETE

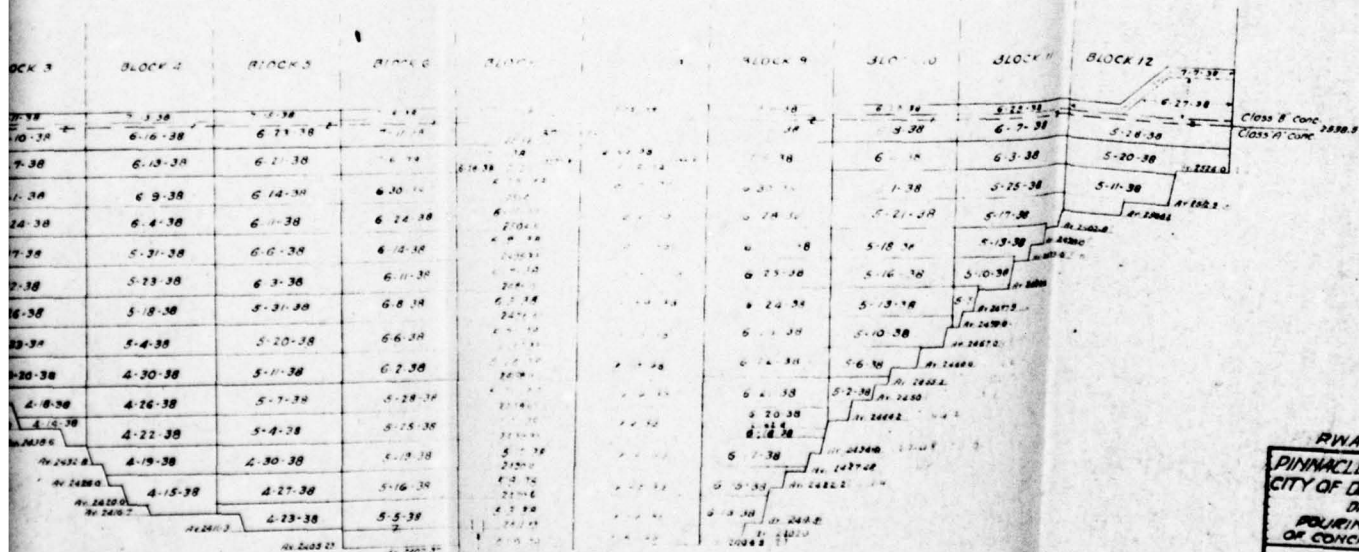
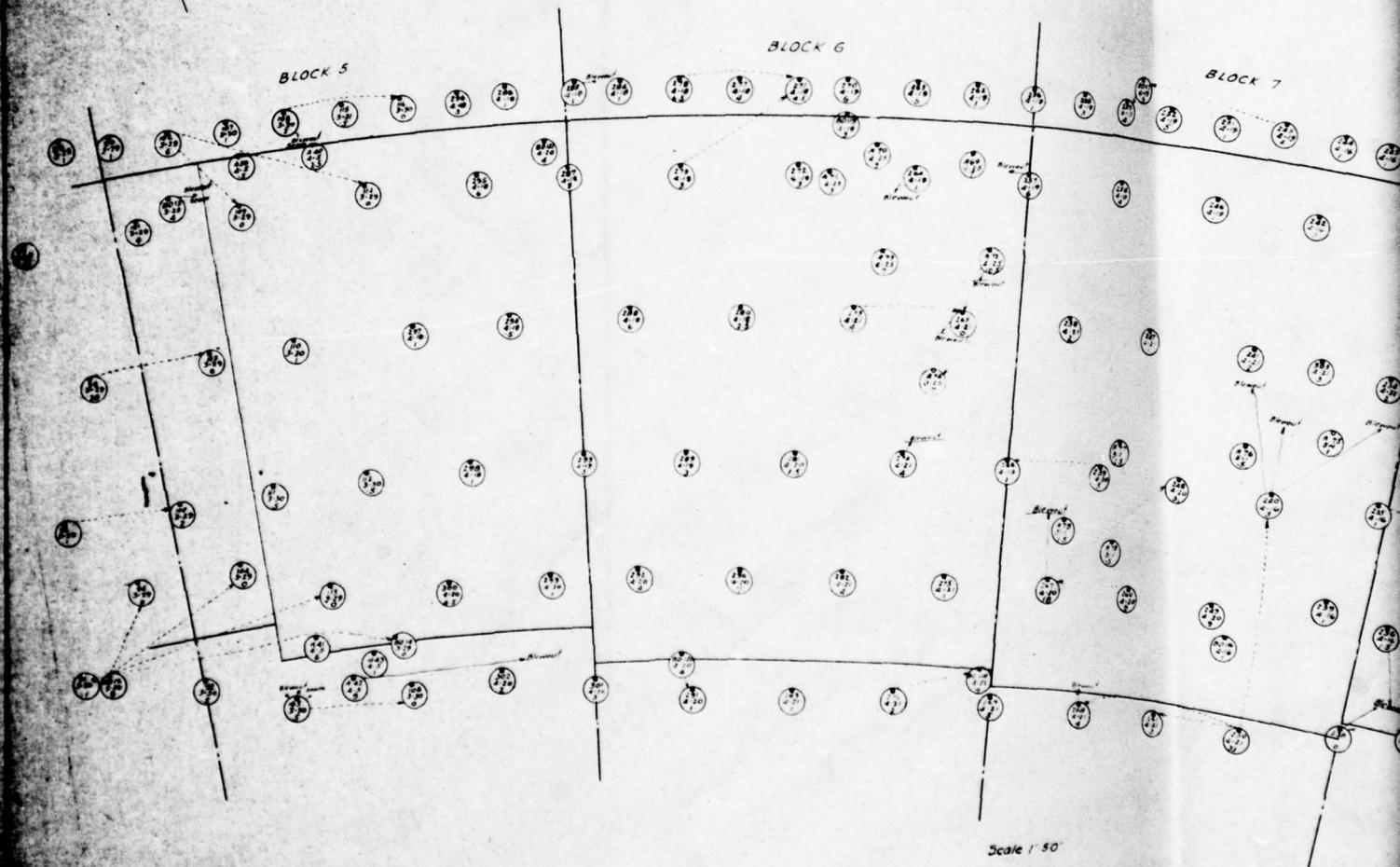
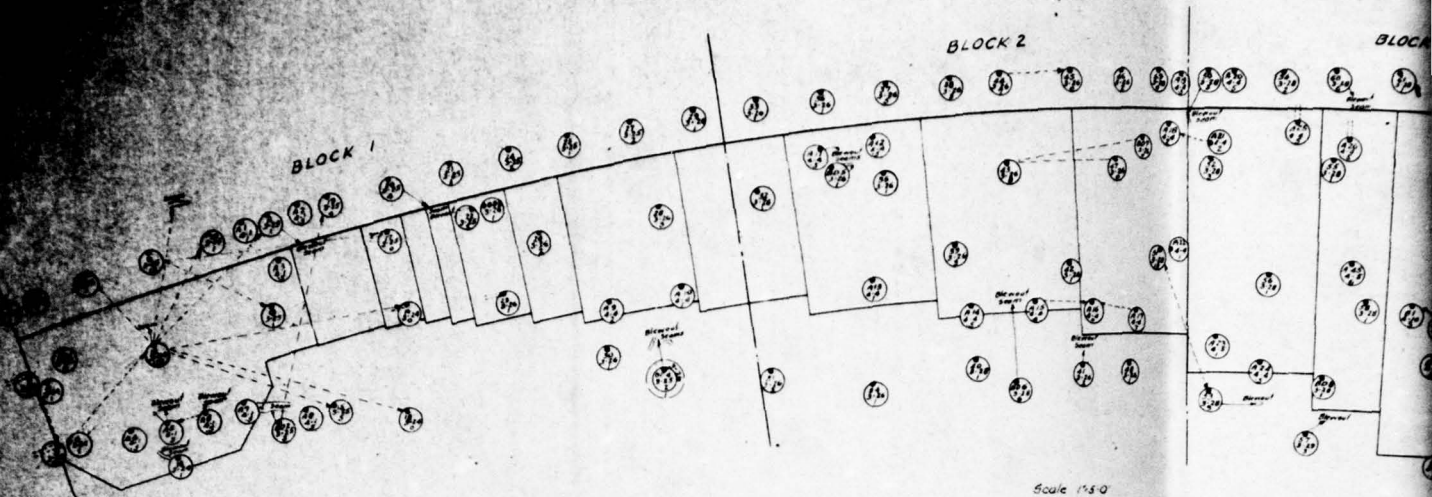
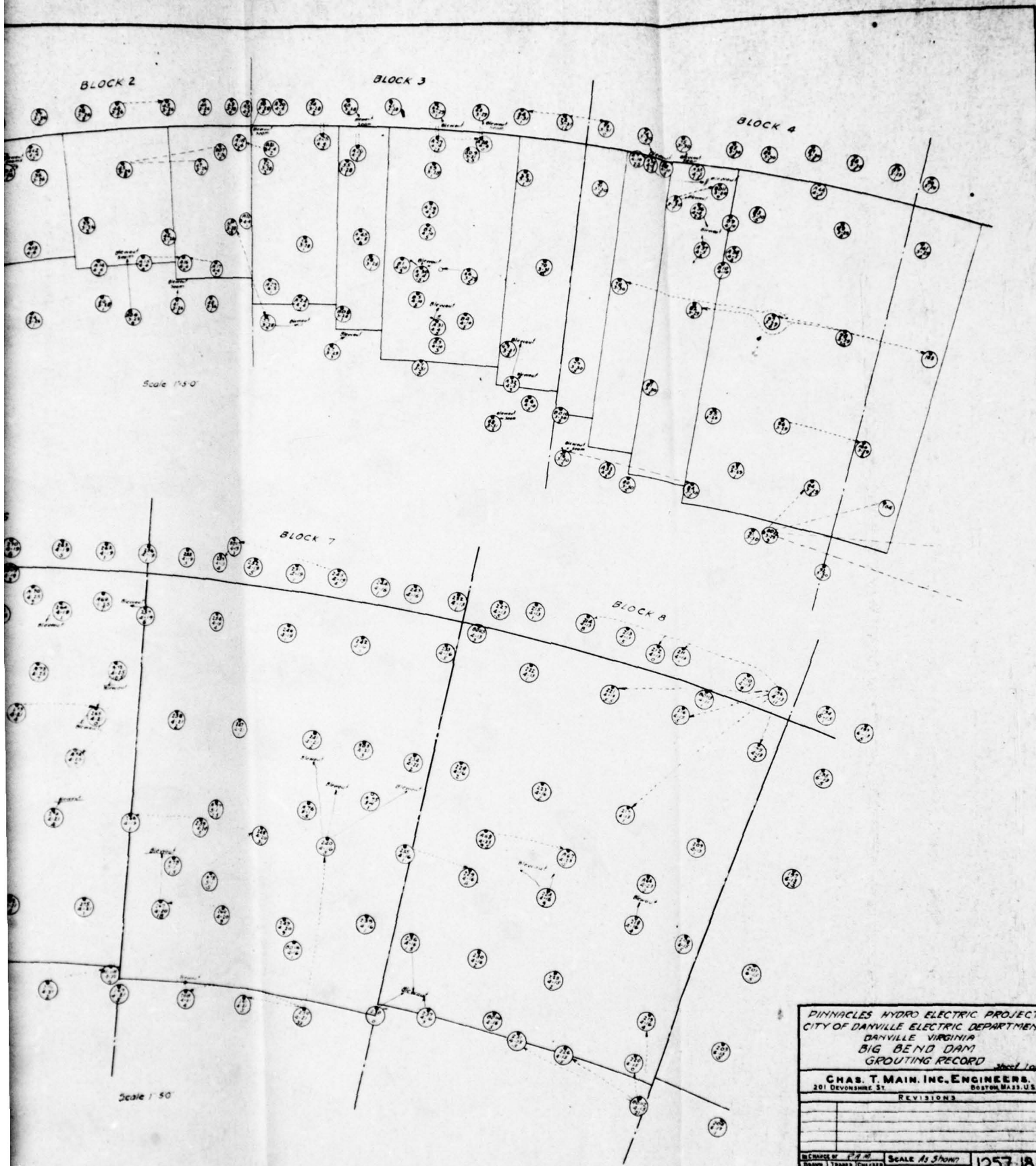


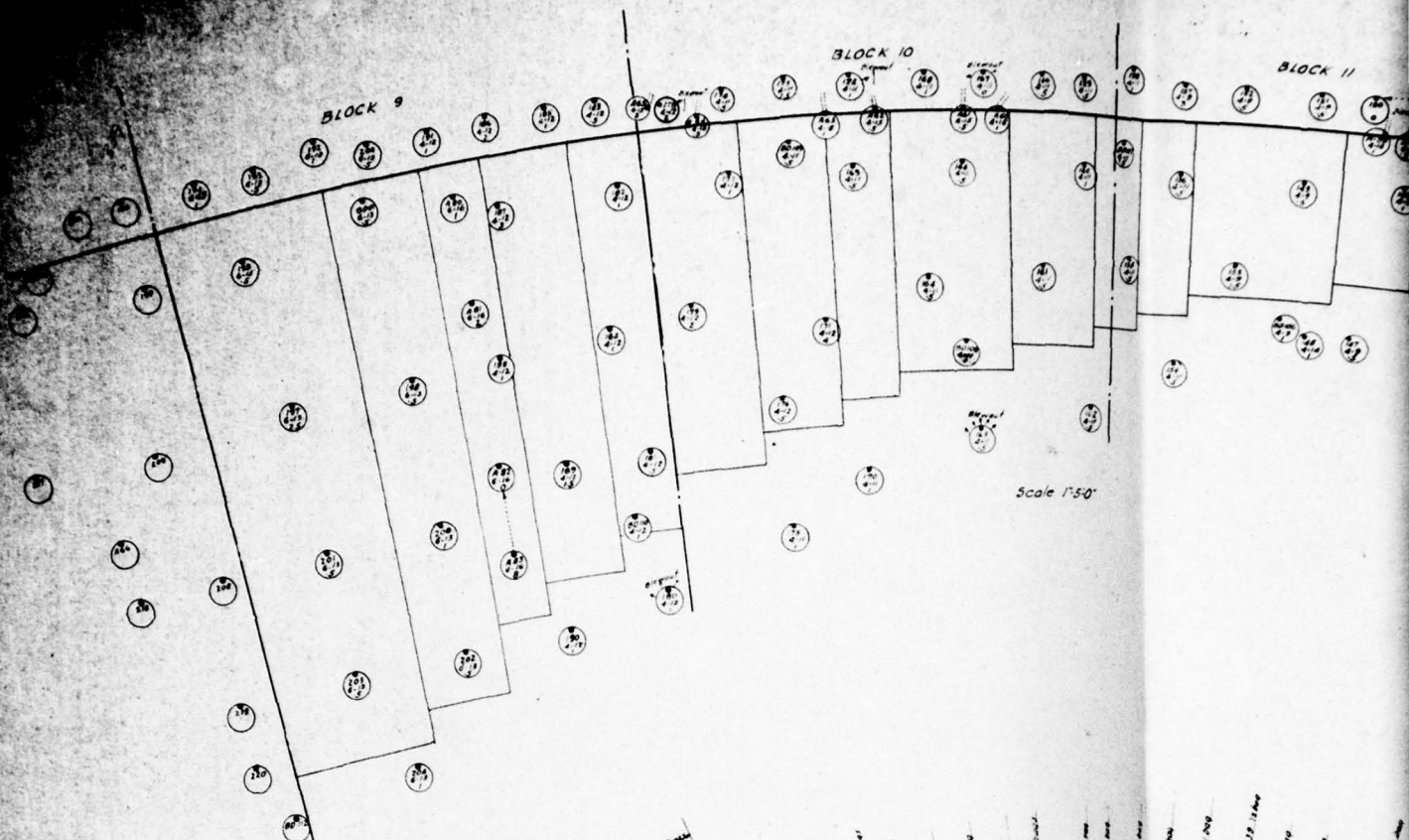
CHART SHOWING POURING DATES AND RISE OF CONCRETE

PINA PROJECT V&H STAR
 PINNACLES HYDRO-ELECTRIC PROJECT
 CITY OF DANVILLE ELECTRIC DEPARTMENT
 DANVILLE, VIRGINIA
 POURING AND TESTING RECORD
 OF CONCRETE IN THE BIG BEND DAM
 CHAS. T. MAIN, INC. ENGINEERS
 201 Devonshire St. BOSTON, MASS. U.S.A.
 REVISIONS
 SCALE 1"=20'
 DATE OCT. 5, 1930
 1257-18



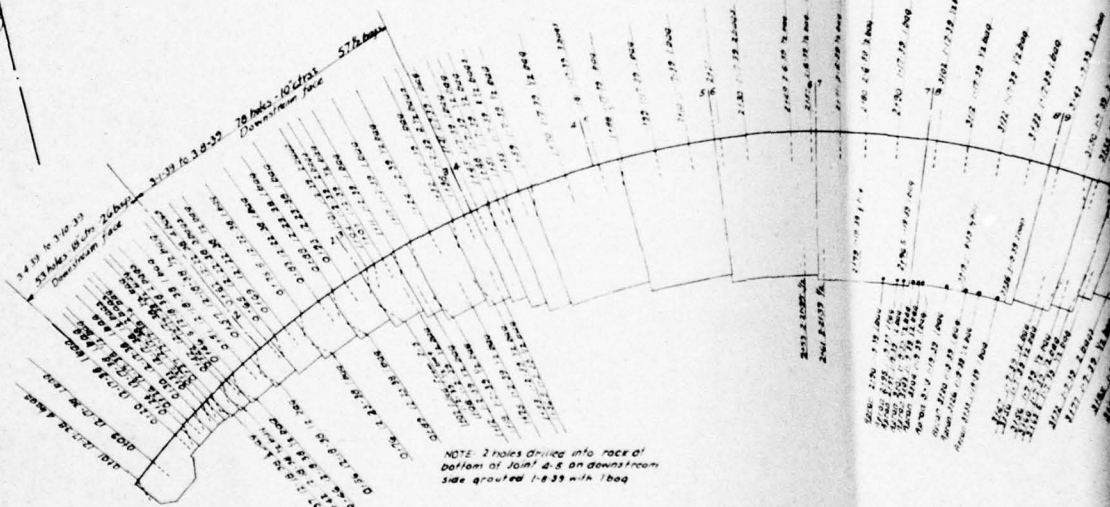


PINNACLES HYDRO ELECTRIC PROJECT			
CITY OF DANVILLE ELECTRIC DEPARTMENT			
DANVILLE VIRGINIA			
BIG BEND DAM			
GROUTING RECORD			
Sheet 1 of 2			
CHAS. T. MAIN, INC. ENGINEERS.			
201 DEVONSHIRE ST. BOSTON, MASS. U.S.A.			
REVISIONS			
DRAWN BY P.M.H.		SCALE AS SHOWN	
CHECKED BY J.W.B.		DATE FEB 4, 1933	
		1257-188	



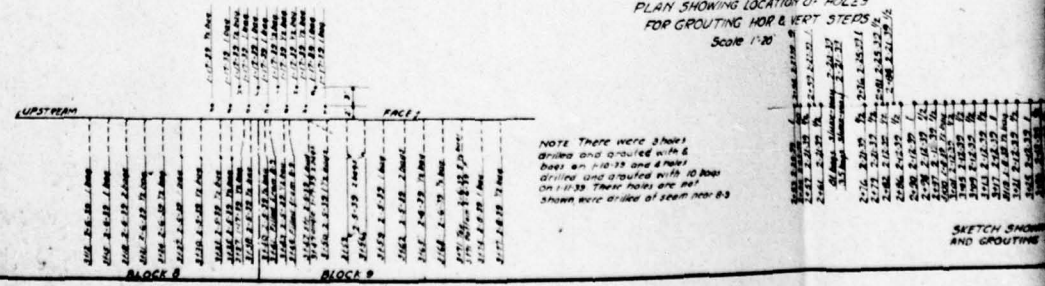
RECORD OF GROUTING VERTICAL JOINTS

JOINT	DATE	REMARKS
1	11-15-35	See 11-15-35 30 Minutes
2	11-15-35	"
3	11-15-35	"
4	11-15-35	"
5	11-15-35	"
6	11-15-35	"
7	11-15-35	"
8	11-15-35	"
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88	11-15-35	"
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91	11-15-35	"
92	11-15-35	"
93	11-15-35	"
94	11-15-35	"
95	11-15-35	"
96	11-15-35	"
97	11-15-35	"
98	11-15-35	"
99	11-15-35	"
100	11-15-35	"

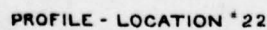
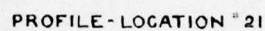


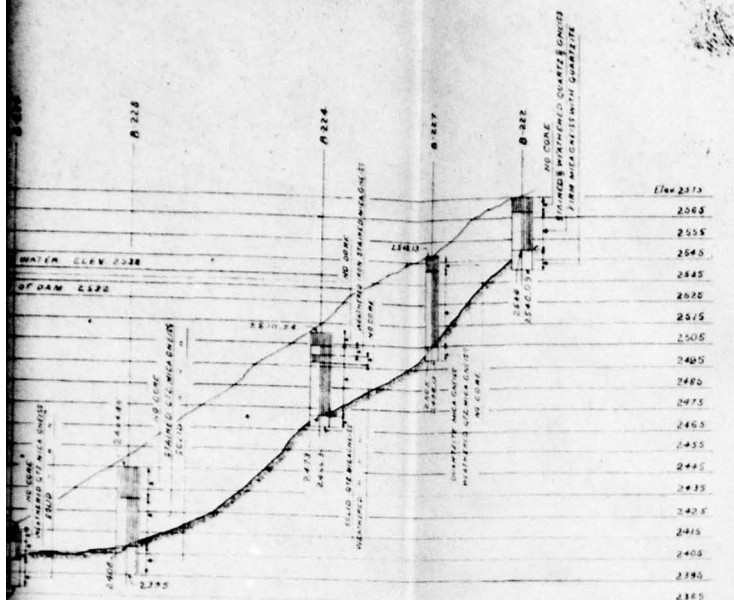
SKETCH SHOWING ADDITIONAL DRILLING AND GROUTING AT JOINT BETWEEN BLOCKS 8 & 9

PLAN SHOWING LOCATION OF HOLES FOR GROUTING HOR & VERT STEPS

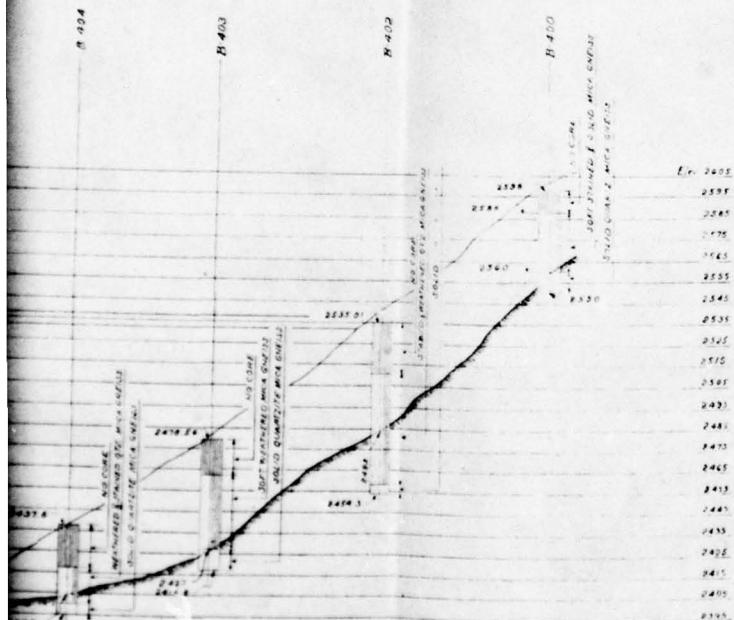


SKETCH SHOWING ADDITIONAL DRILLING AND GROUTING

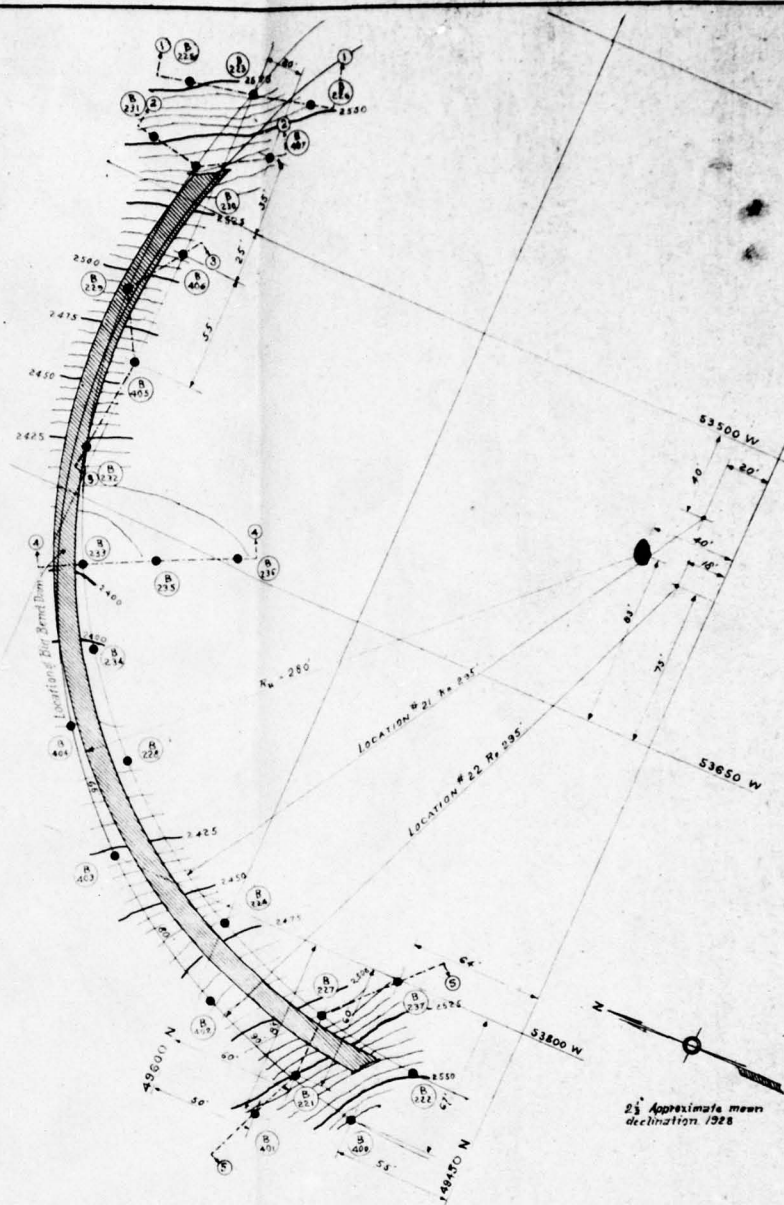




LOCATION # 21



LOCATION # 22



PLAN

SHOWING SOUND ROCK CONTOURS
Assumed for Design. Actual Sound rock elevations
to be determined by inspection of foundation
excavation by the Engineer

Note:
For additional sections see sheet 13

LEGEND

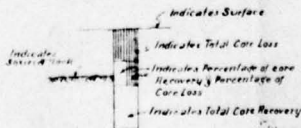


PLATE 7

PWA DOCKET # 1151 - PM
SHEET 12 OF 13 SHEETS

PINNACLES HYDRO-ELECTRIC PROJECT

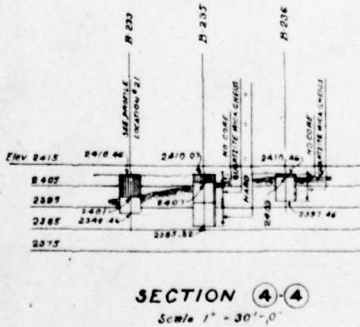
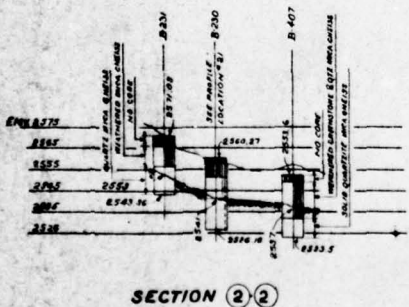
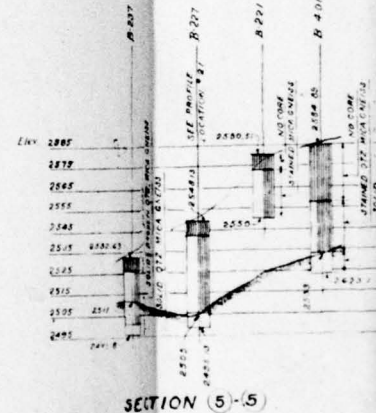
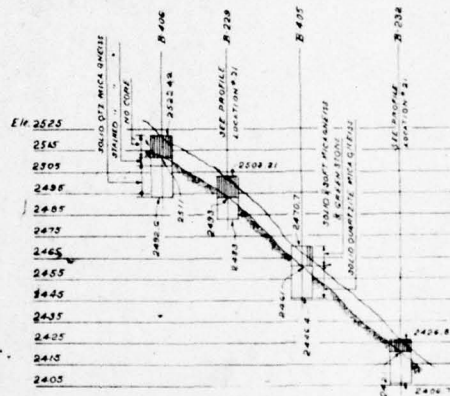
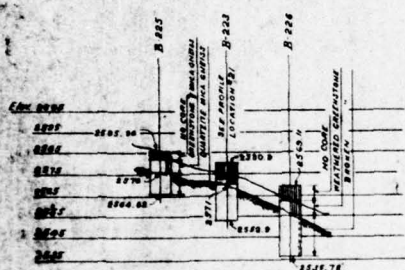
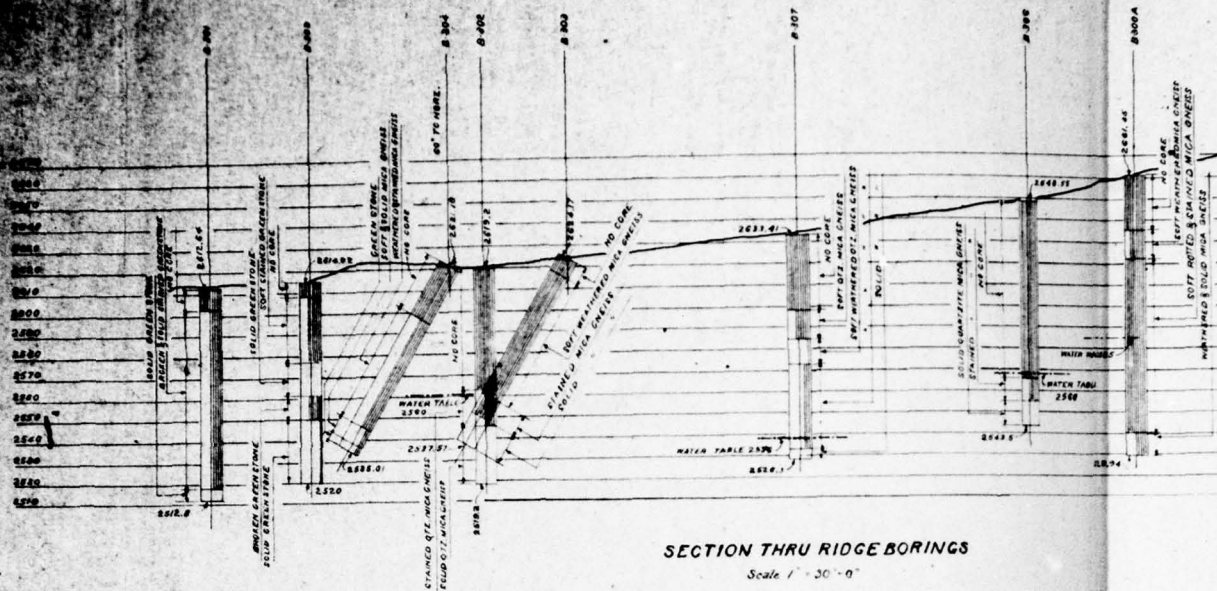
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
BIG BEND DAM

DIAMOND DRILL CORE BORINGS

CHAS. T. MAIN, INC. ENGINEERS
201 DEVONSHIRE ST. BOSTON, MASS. U.S.A.

REVISIONS

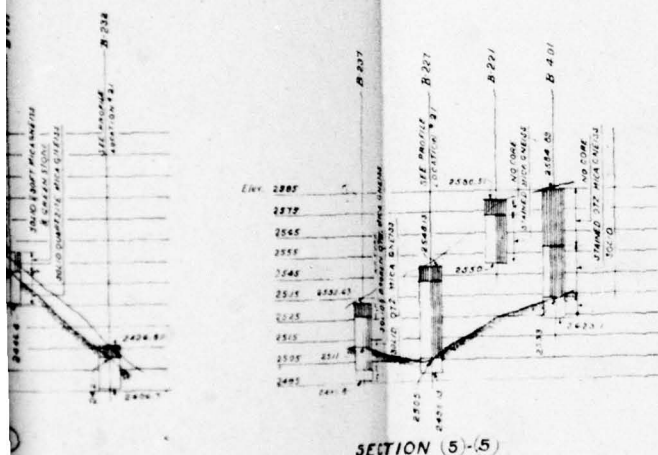
DATE: 10/1/34 SCALE: 1" = 20' DRAWN BY: J.C.M. CHECKED BY: J.C.M. DATE: 10/1/34



For location of
Sections 1-1, 2-2, 3-3, 4-4,
and 5-5 see sheet 12

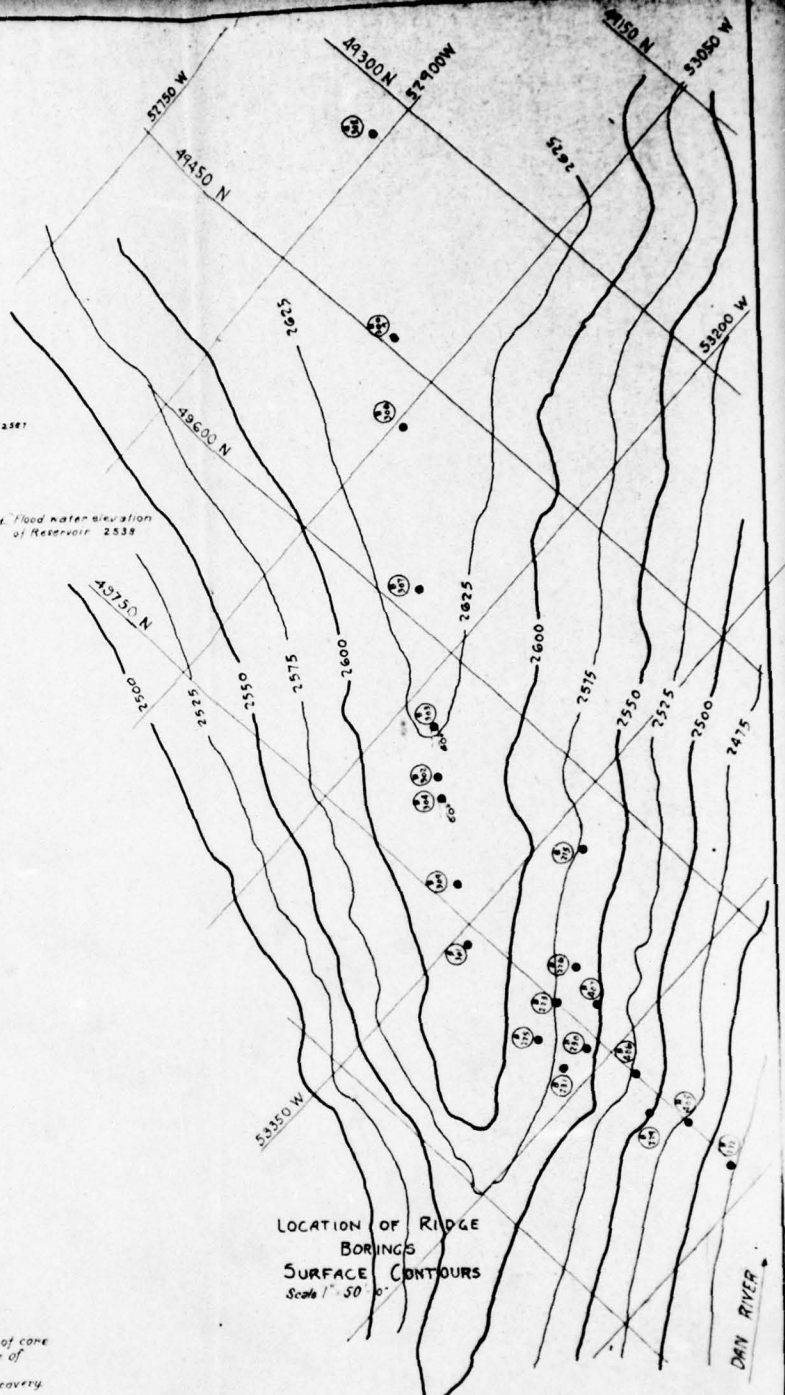
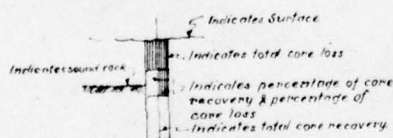
THRU RIDGE BORINGS

Scale 1" = 30' - 0"



SECTION (5)-(5)

LEGEND



LOCATION OF RIDGE BORINGS
SURFACE CONTOURS
Scale 1" = 50' - 0"

SHEET 13 OF 13 SHEETS

PLATE 8

PINNACLES HYDRO-ELECTRIC PROJECT

CITY OF DANVILLE ELECTRIC DEPARTMENT

DANVILLE VIRGINIA

BIG BEND DAM

DIAMOND DRILL CORE BORINGS ON RIDGE

CHAS. T. MAIN, INC. ENGINEERS

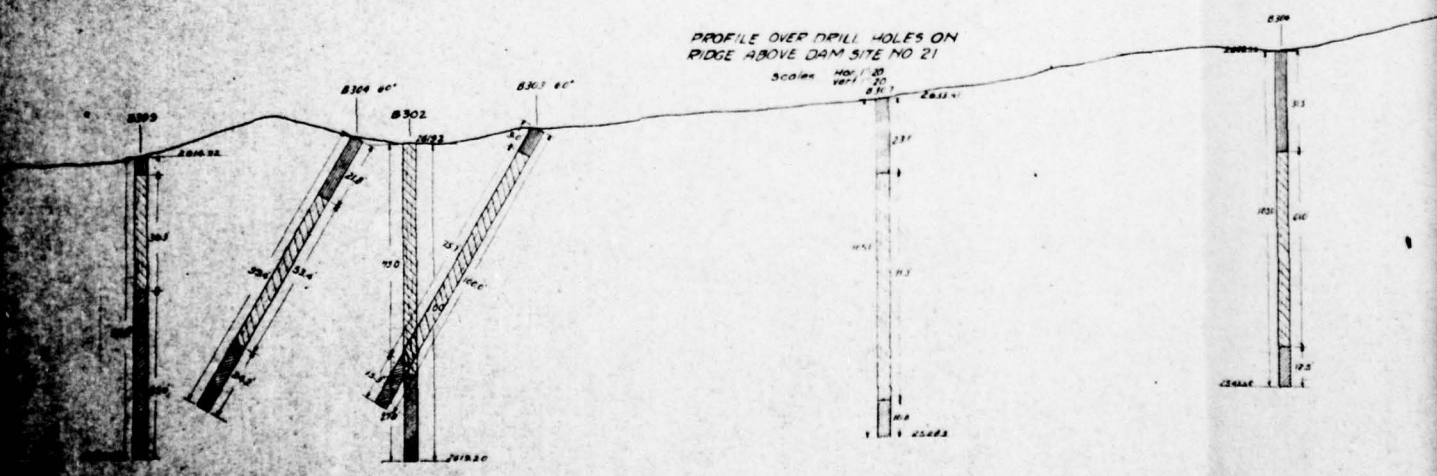
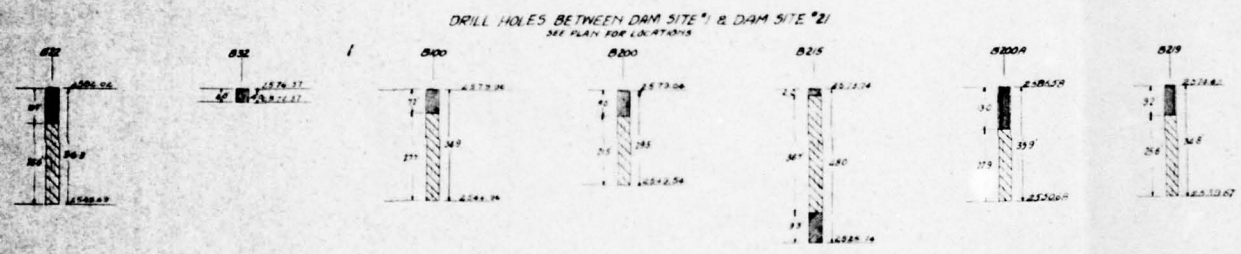
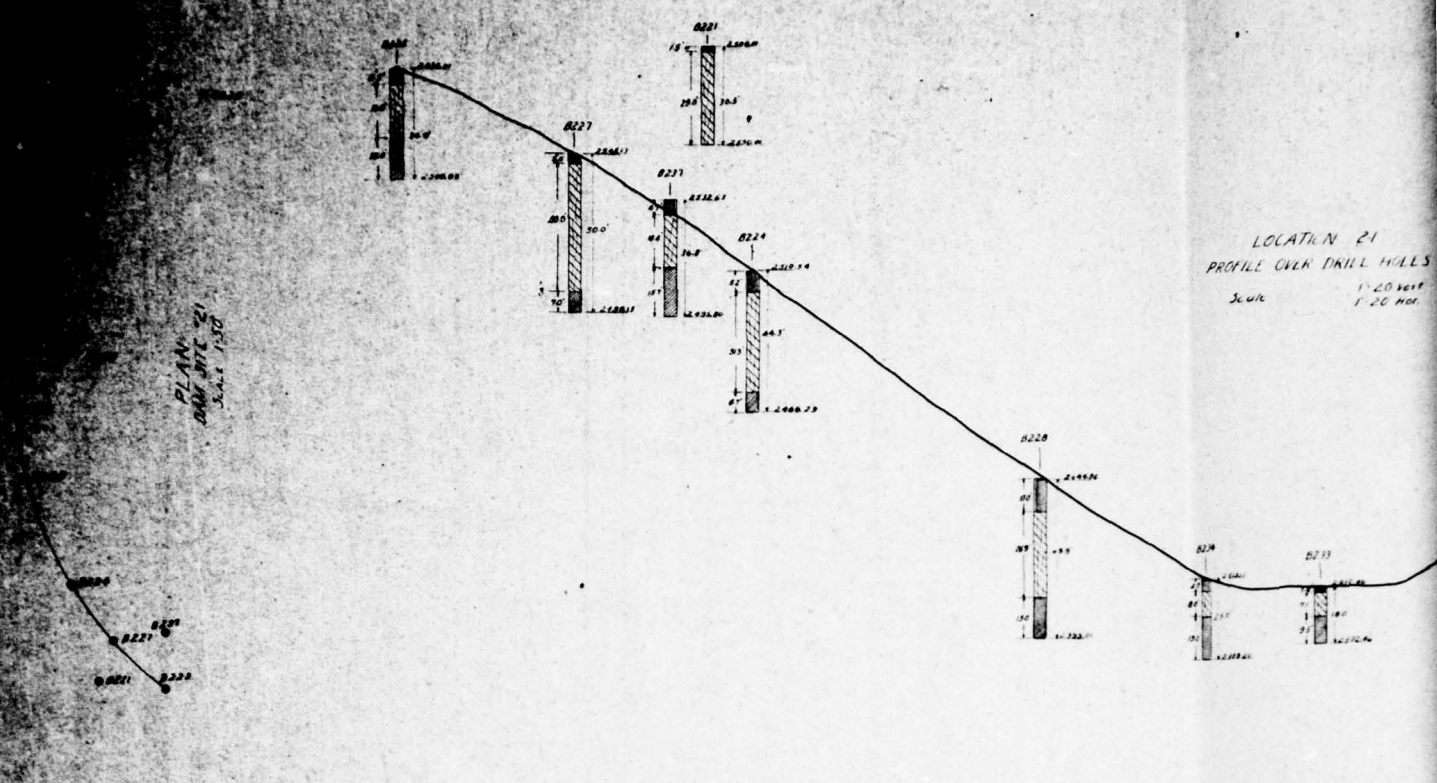
201 DEVONSHIRE ST. BOSTON MASS. U.S.A.

REVISIONS

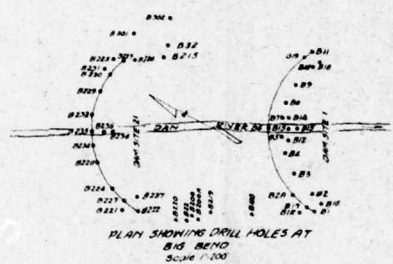
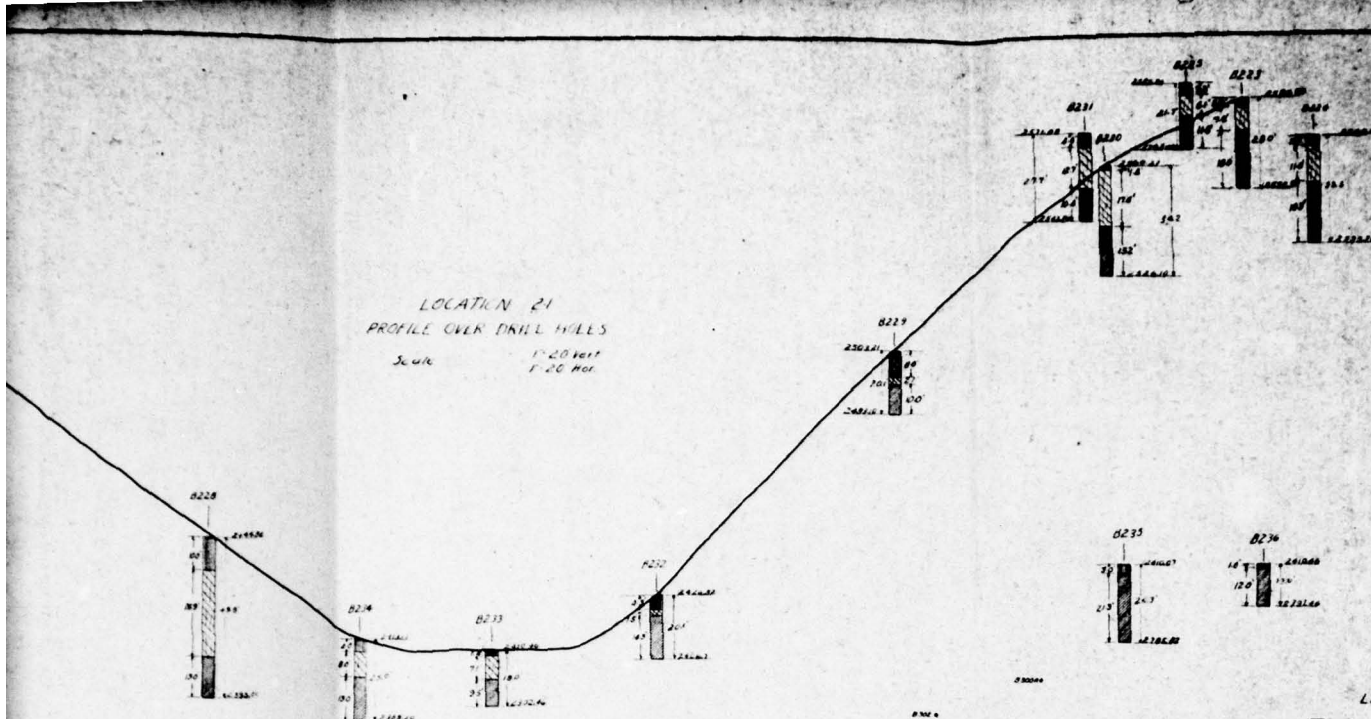
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REVIEWED BY: [Signature] DATED: [Date]
DRAWN BY: [Signature] DATED: [Date]
CHECKED BY: [Signature] DATED: [Date]
DATE: Aug 30, 1937

1247-713



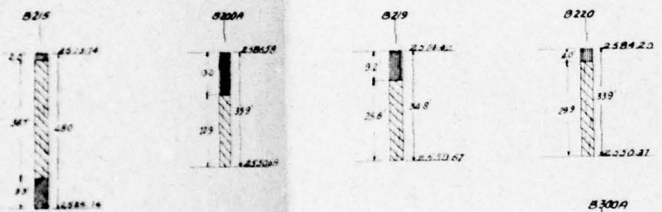
LOCATION 21
 PROFILE OVER DRILL HOLES
 Scale 1" = 20 feet
 1" = 20 feet



LEGEND

- Loose material
- Weathered rock
- Solid rock suitable for foundation

DAM SITE #21



DRILL HOLES ON SITE NO 21

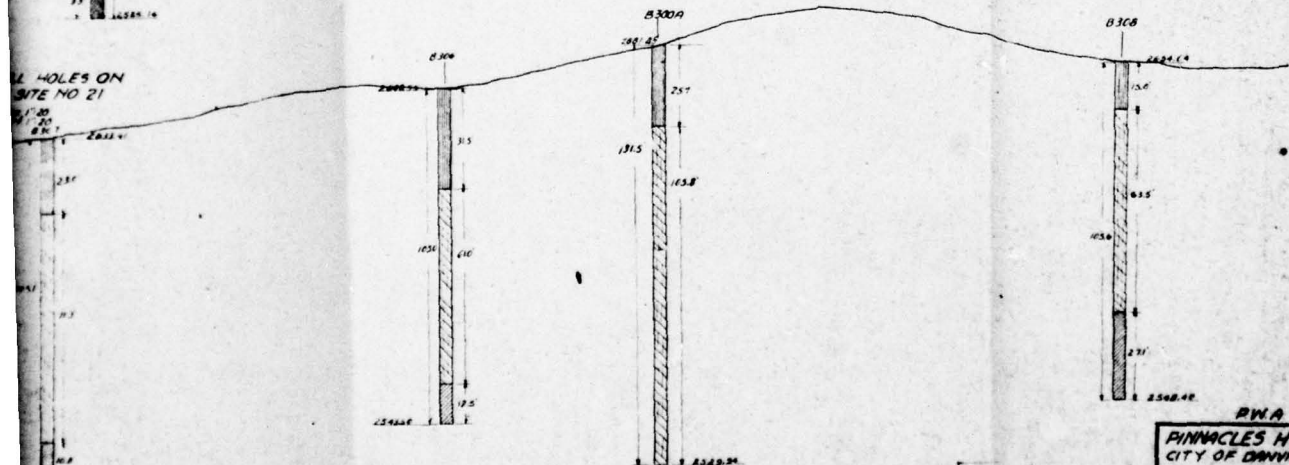


PLATE 9

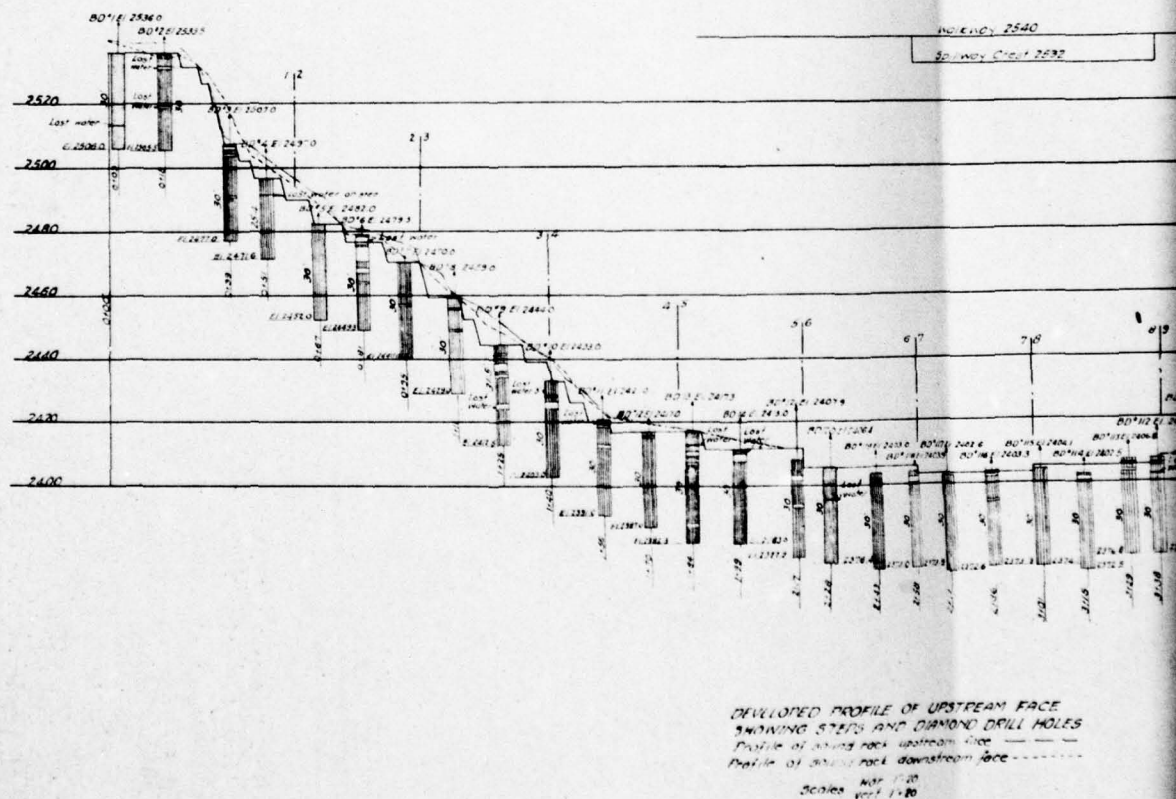
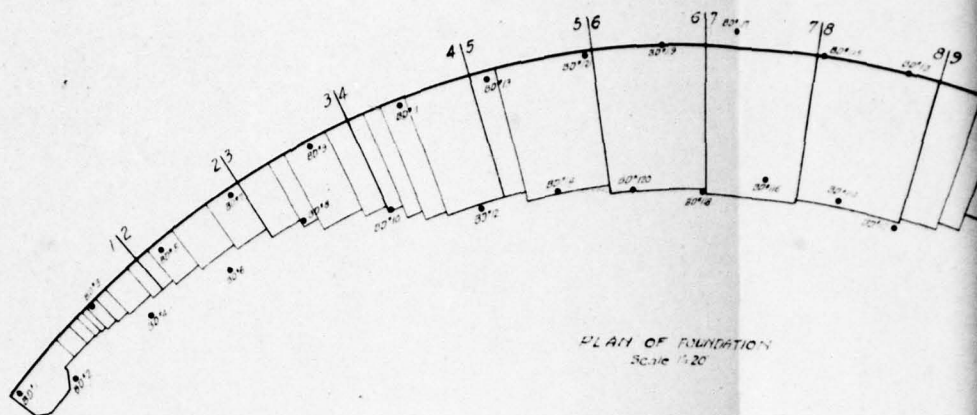
PWA PROJECT 151/R

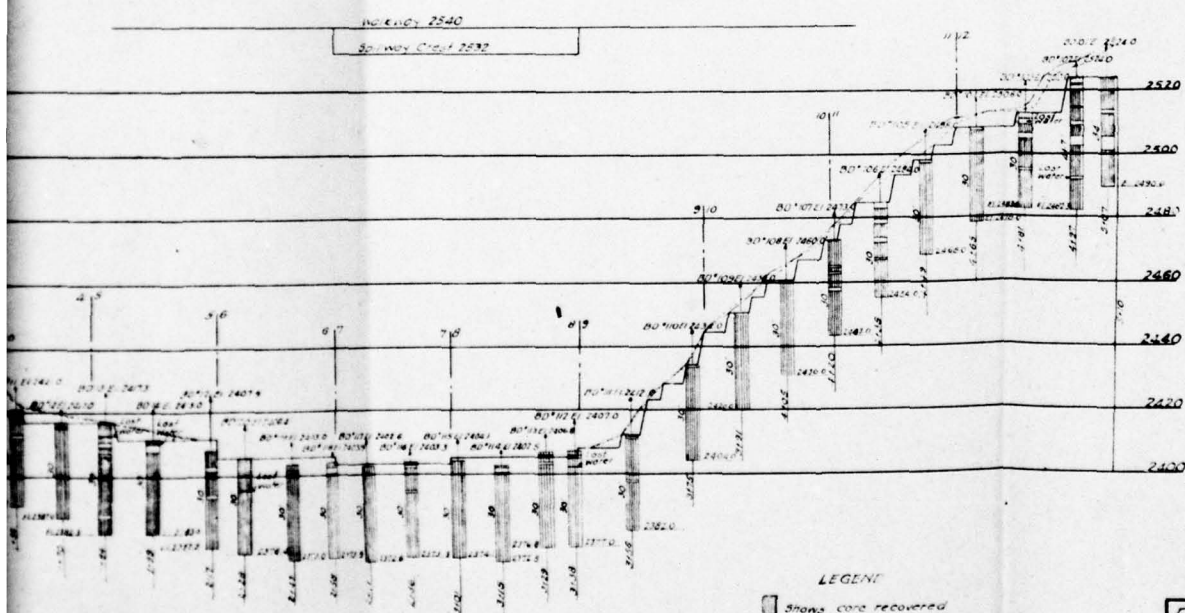
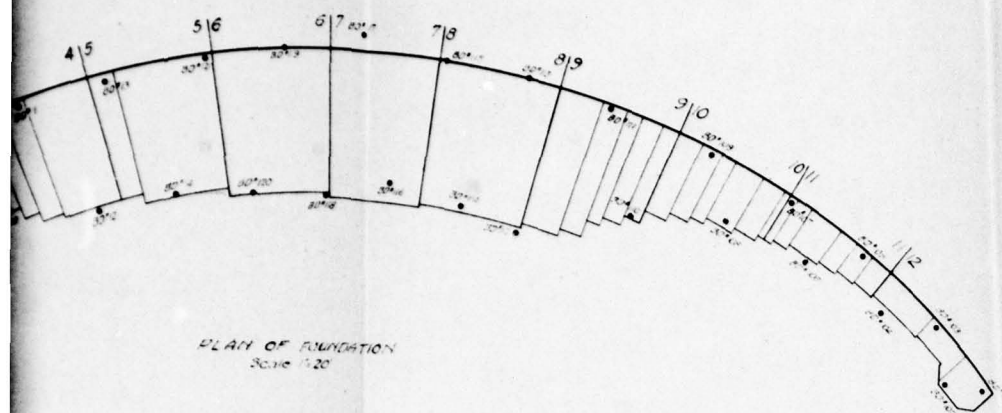
PINNACLES HYDRO-ELECTRIC PROJECT
 CITY OF DANVILLE ELECTRIC DEPARTMENT
 DANVILLE, VIRGINIA
BIG BEND COPE BORINGS
 SHEET 4

CHAS. T. MAIN, INC. ENGINEERS
 214 FARMER BUILDING
 DANVILLE, VIRGINIA

REVISIONS	

DESIGNED BY: [] CHECKED BY: []
 DRAWN BY: [] DATE: []

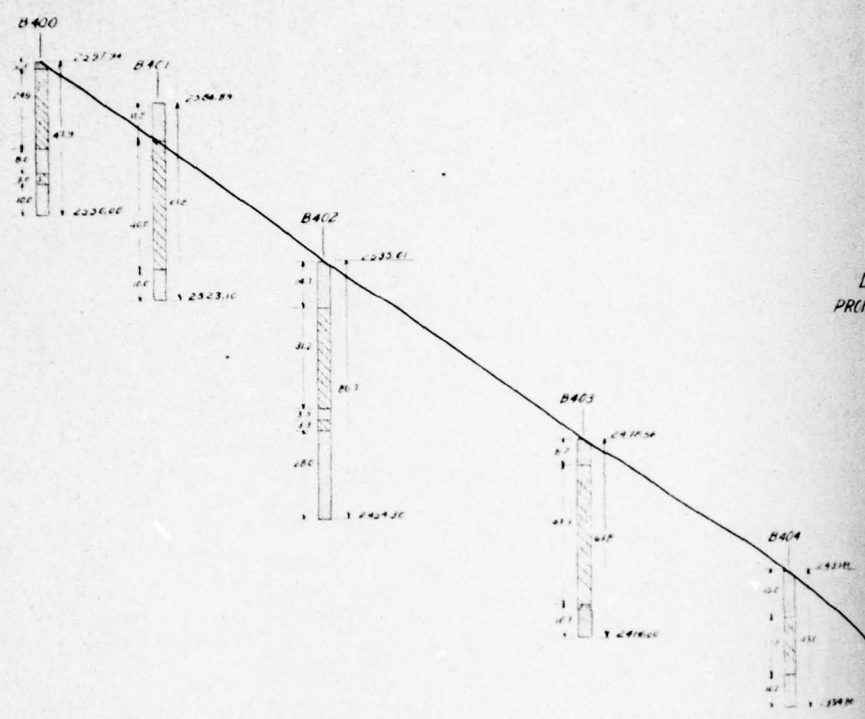
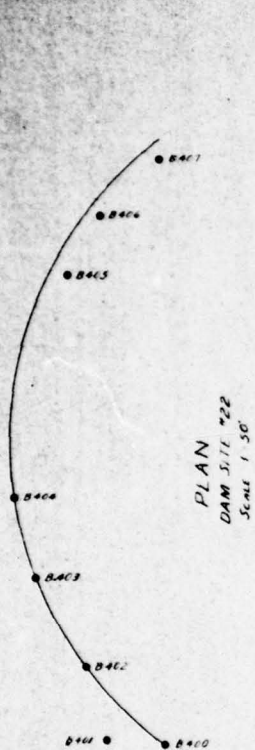




DEVELOPED PROFILE OF UPSTREAM FACE
SHOWING STEPS AND DIAMOND DRILL HOLES
Profile of sound rock upstream face — — —
Profile of jointed rock downstream face - - - - -

Series No. 120
Vol. 1-20

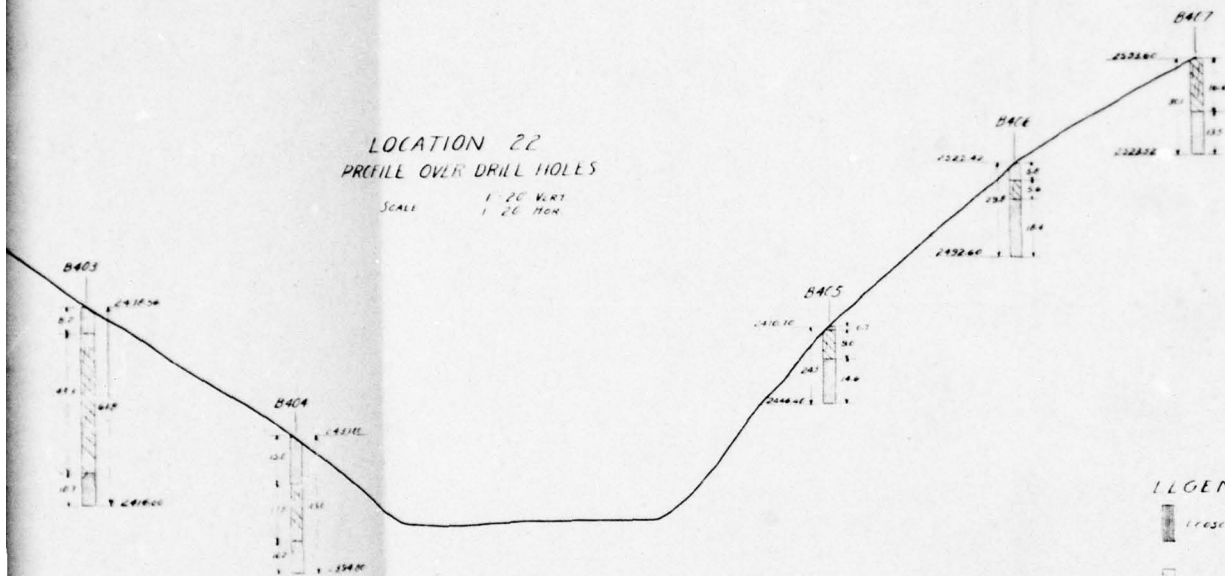
PINNACLES-HYDRO ELECTRIC PROJECT					
CITY OF DANVILLE ELECTRIC DEPARTMENT					
DANVILLE VIRGINIA					
BIG BEND DAM					
FOUNDATION AND GROUT HOLES					
CHAS. T. MAIN, INC. ENGINEERS					
201 DEVONSHIRE ST.			BOSTON MASS. U.S.A.		
REVISIONS					
REQUIRE BY 7-2-87		SCALE AS SHOWN		1257-107	
TOTAL CHECKED J.C.B. J.L.B.		DATE AUG 8 1978			



LOCATION 22
PROFILE OVER DRILL HOLES
SCALE 1" = 20' VERT
1" = 20' HORIZ

LOCATION 22
PROFILE OVER DRILL HOLES

SCALE 1" = 20' VERT
1" = 20' HOR



LEGEND

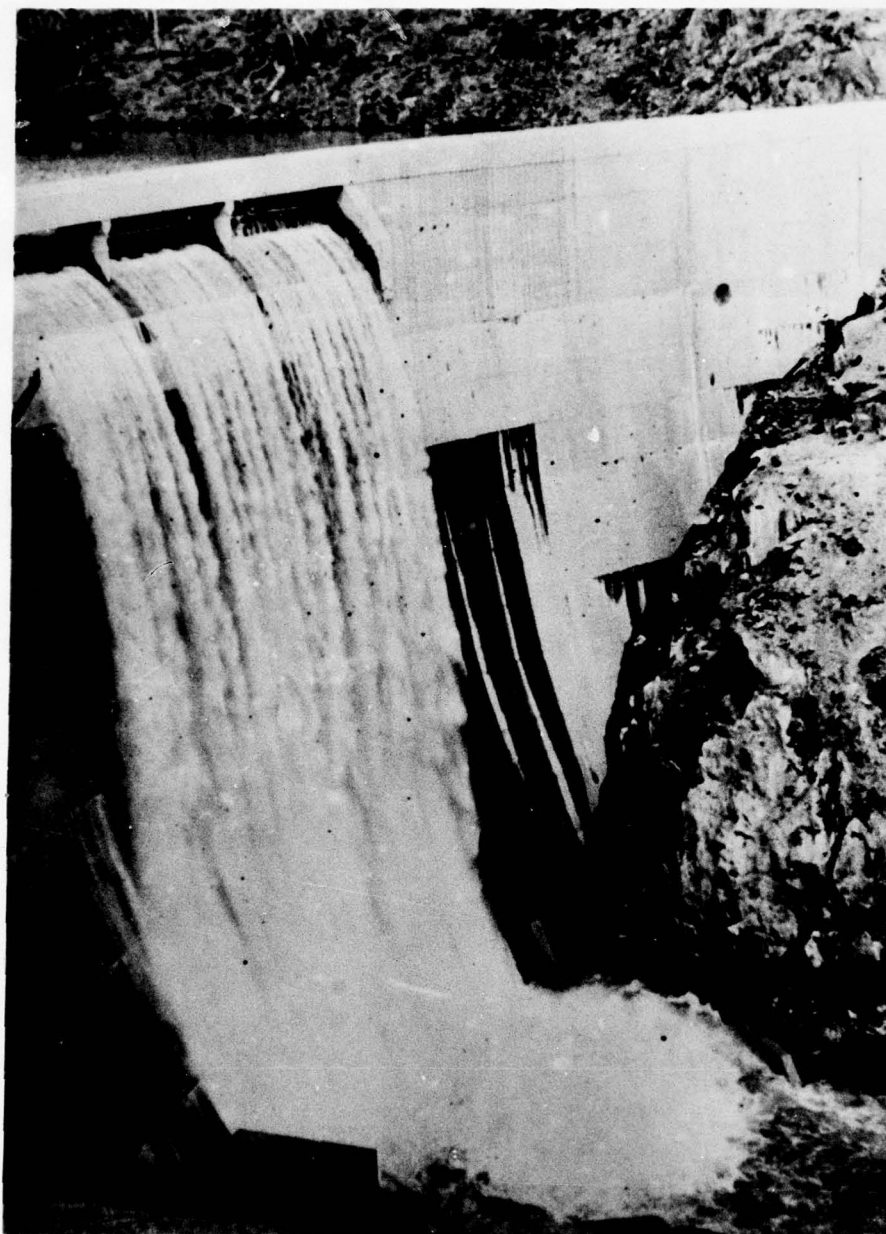
- LOOSE MATERIAL
- WEATHERED ROCK
- SOLID MATERIAL FOR FOUNDATION

PWA PROJECT 1151R			
PINNACLES HYDRO-ELECTRIC PROJECT			
CITY OF DANVILLE ELECTRIC DEPARTMENT			
DANVILLE, VIRGINIA			
BIG BEND CORE BORINGS			
SHEET 5			
CHAS. T. MAIN, INC. ENGINEERS			
314 MUNICIPAL BUILDING DANVILLE, VIRGINIA			
REVISIONS			
IN CHARGE BY R. A. M.		SCALE AS SHOWN	
DRAWN BY J. D. F.		DATE JULY 28, 1936	
		1257-177	

PLATE II

APPENDIX II

PHOTOGRAPHS



TALBOTT DAM---PHOTO PRIOR TO 1940
NOTE SEEPS THROUGH DAM

PHOTO 1



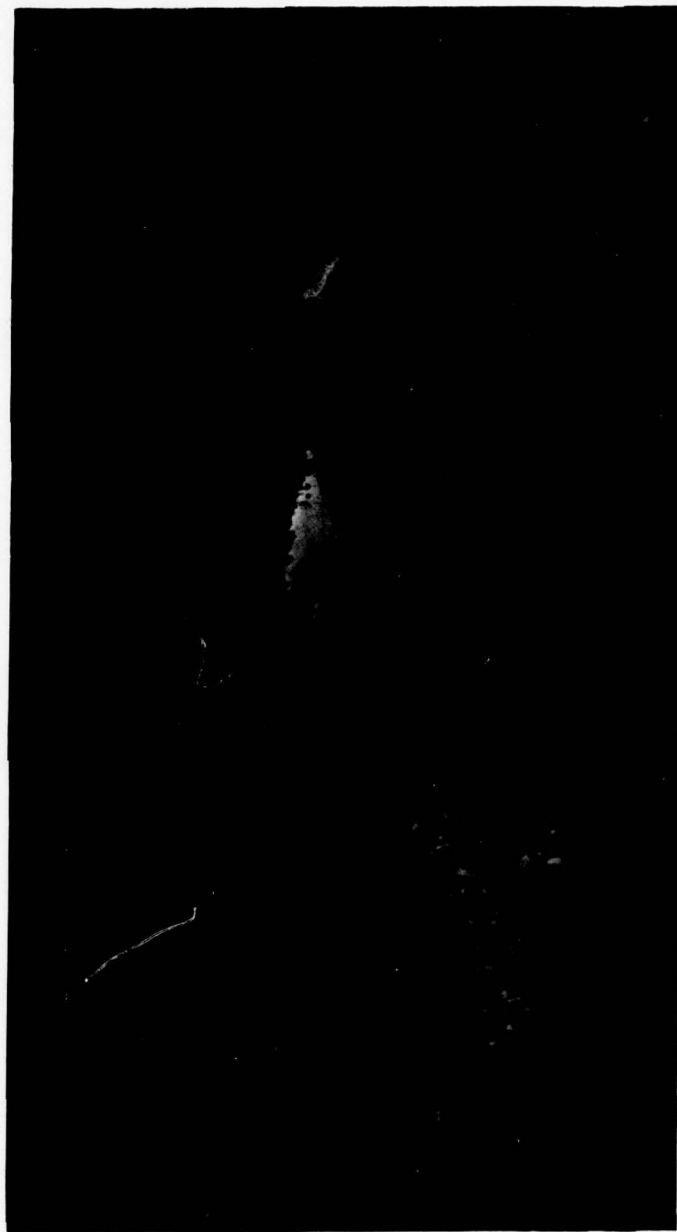
TALBOTT DAM---TAKEN FROM STREAM
BED DOWNSTREAM FROM DAM-NOTE WHITE CALCIUM
DEPOSITS FROM SEEPAGE THROUGH CONSTRUCTION JOINTS

PHOTO 2



TALBOTT DAM-VIEW OF VALUE
HOUSE AND STILLING BASIN TAKEN FROM
THE TOP OF THE DAM

PHOTO 3



TALBOTT DAM-VIEW DOWNSTREAM
TAKEN FROM TOP OF DAM

PHOTO 4

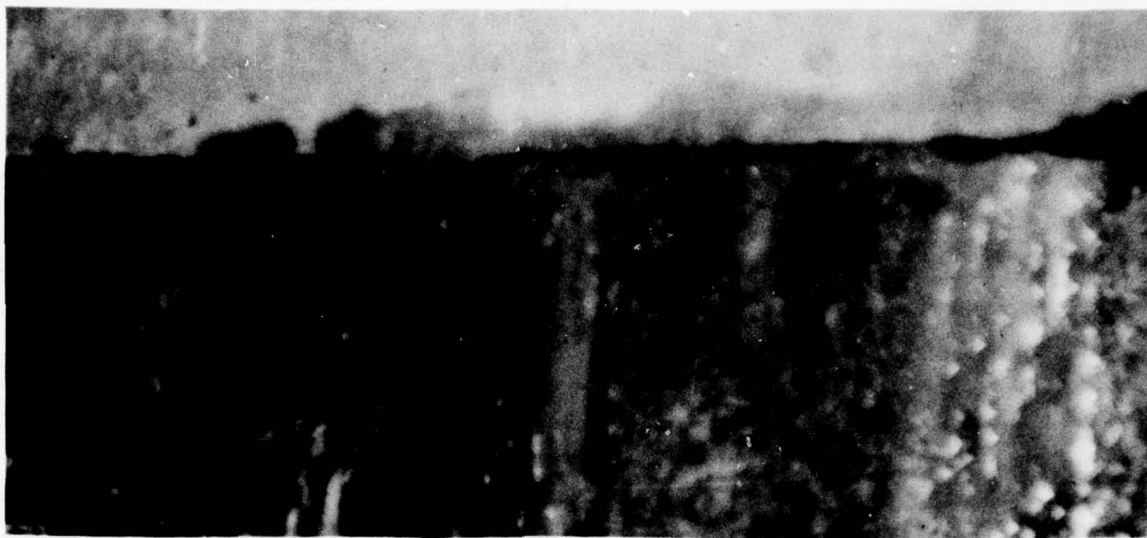
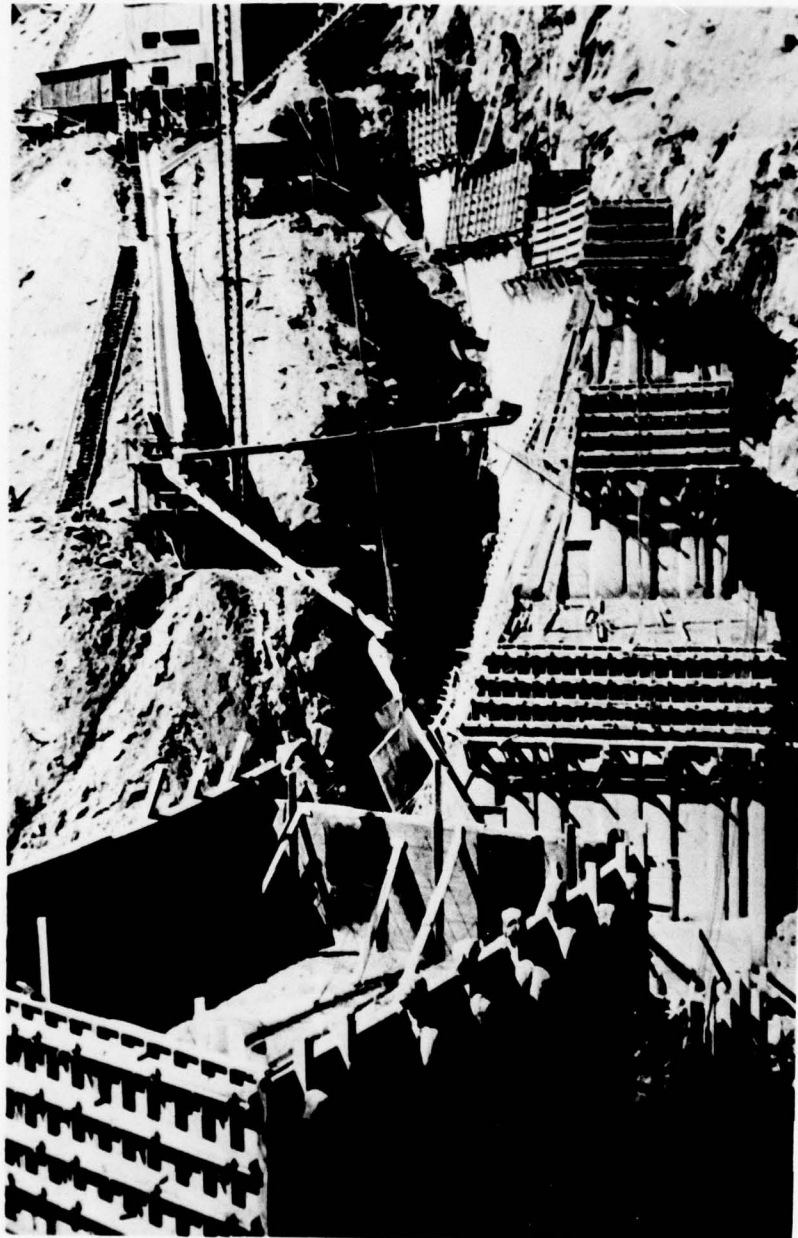


PHOTO 5



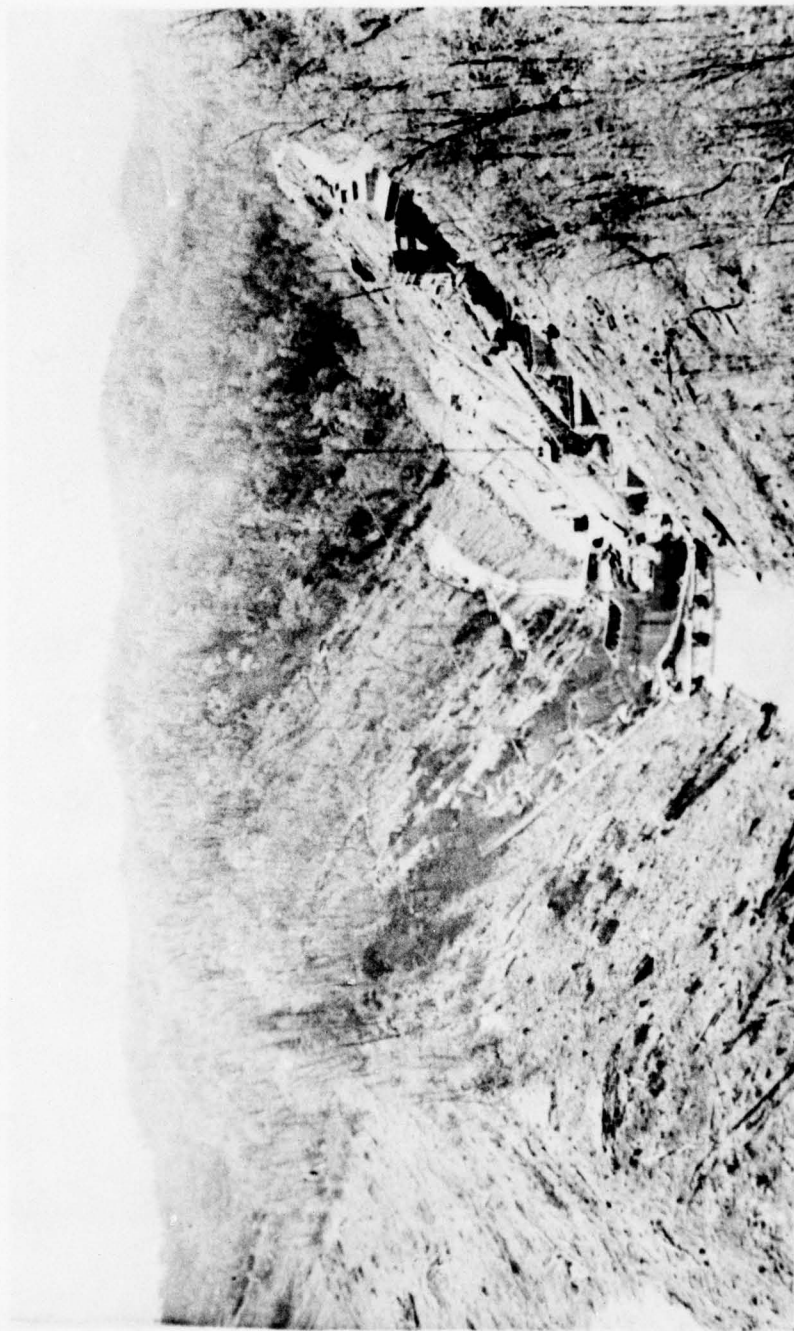
PHOTO 6

TALBOTT DAM-CRACK AT LEFT ABUTMENT
EXTENDING 30 FT. FROM ABUTMENT/ROCK CONTACT
4 LIFTS BELOW WALKWAY



TALBOTT DAM--RIGHT ABUTMENT
DURING CONSTRUCTION

PHOTO 7



TALBOTT DAM---UNDER CONSTRUCTION
LOOKING DOWNSTREAM

PHOTO 8

APPENDIX III
FIELD OBSERVATIONS

APPENDIX III

PHASE I - FIELD OBSERVATIONS

Name of Dam: Talbott Storage Dam (VA 14101)
County: Patrick State: Virginia
Coordinates: Lat. 3640.6 Long. 8023.8
Date of Inspection: 27 July 1978
Weather: Partly cloudy
Temperature: 74°F
Pool Elevation at Time of Inspection: 2527.2 MSL
Tailwater at Time of Inspection: 2410 MSL

Norfolk District Inspection Personnel:

Jeff Irving, Structural Engineer
Meade Stith, Structural Engineer
Lonnie Baird, Electrical Engineer
Henry Hammond, Mechanical Engineer
Bill Barker, Geologist
Jim Robinson, Hydrologist
Rob Schonk, Geotechnical Engineer (Recorder)

Other attendees:

Sam Stryker, City of Danville
Dave Lucado, State Water Control Board
Clyne Willis, Supervisor of the Pinnacles Hydro-Electric Plant,
City of Danville
Jesse Thompson, Talbott Dam Patrolman
John Bowman, Townes Dam Patrolman

1. GENERAL: At the time of the inspection, the level of the water in the reservoir was 7.8 feet below the level of the spillway. No water was passing through the outlet works during the inspection.

2. CONCRETE ARCH DAM

2.1 Seepage: Seepage was noted on the face of the dam through both horizontal and vertical construction joints. Many of these seeps showed very heavy calcium deposits. The left abutment appeared to be wetter than the right.

2.2 Structure to Abutment Junction: There appeared to be some leakage along the contact joints. A spring was found approximately 150 feet downstream on the left abutment approximately 50' below the top of the dam. Much of the contact area was obscured by humus material that has been deposited since the dam was constructed.

2.3 Drains: No drains were observed during the inspections.

2.4 Water Passages: Besides the ungated spillway, the water passages include a bubbler connected by a 36-inch line, two 6-inch bypass lines and a 6-inch drain.

2.5 Foundation: The foundation of the dam along both abutments and channel section is fairly competent quartz, mica gneiss. The strike of the schistosity was measured at N 45° to 60° E with a dip of 60° to 80° to the southeast.

2.6 Concrete Surfaces: A horizontal crack, approximately 1/2 inch wide and 30 feet long, is at the bottom of the fourth lift from the top beginning at the left abutment on the downstream face. The concrete above the crack is displaced approximately 1/8 inch. The crack is seeping at a rate estimated to be 1 GPM.

2.7 Vertical and Horizontal Alignment: Small displacements (less than 1/4-inch) were noted on each side of the walkway passing over the spillway.

2.8 Monolith Joints: There appeared to be some leakage and leaching in the monolith joints.

2.9 Construction Joints: The vertical construction joints appear open, however, drawings indicate the joints were grouted.

3. OUTLET WORKS:

3.1 Intake Structure: The intake structure is submerged for normal pool elevation. It consists of a 72-inch diameter intake which transitions in a conical fashion to a 36-inch pipeline. The intake is protected by a trash screen and includes a stop gate.

3.2 Outlet Structure: The outlet structure consists of a bubbler which serves as an energy dissipator. It is approximately 20 feet in diameter and consists of 26 equally spaced nozzles.

3.3 Outlet Channel: The outlet channel consists of a concrete apron surrounding the bubbler and the natural stream bed downstream of the apron. The apron also serves to prevent erosion from the spillway discharge flow. There are two stilling basin wall drain holes in the concrete weir that separate the basin from the natural channel. The left drain was open while the right drain was partially clogged with debris. There was no discharge from the dam but was an estimated 15 GPM flow in the downstream channel bed by seeps already noted.

3.3 Emergency Gate: A stopgate is located on the upstream face of the dam. It is controlled by a cable from the machinery shack located at the top of the dam at its center. The emergency gate was

last used four years ago when it was necessary to overhaul one of the valves on the outlet pipe.

4. UNGATED SPILLWAY:

4.1 Concrete Weir: The concrete weir appeared to be in good condition with no apparent spalls or cracks. Conversations with the dam operators indicated that flow over the spillway occurs on the average of once a year.

4.2 Bridge and Piers: The parapet walls and walkway appeared to be in good condition with no signs of spalling or cracking of concrete.

5. INSTRUMENTATION

5.1 Monumentation/Surveys: There is a surveying platform and monument approximately 260 feet downstream of the face of the dam. Eyebolts have been placed in the face of the dam for measuring deflection. Pipes extend into the dam for measuring temperature.

6. RESERVOIR

6.1 Slopes: The reservoir slopes are steep (30° or greater) since the reservoir is sited in a gorge.

6.2 Sedimentation: Conversation with the engineers representing the City of Danville indicated some sedimentation is occurring along the base of the dam. An underwater inspections of the dam was performed by Logan Diving, Inc. of Jacksonville, Florida on 4 and 5 April 1978. A copy of this report is included in Appendix C.

6.3 Water Levels: There is a staff gage at the dam and the water levels are recorded every day at 7:30 AM and 3:30 PM.

7. DOWNSTREAM CHANNEL

7.1 Conditions: The stream bed itself is approximately 50 feet wide. The sides rise steeply to about five feet above stream elevation and plateaus out for from 10 to 15 feet. From this point they rise steeply once again. Above the small plateaus, the slopes are wooded; below them the slopes are grassy.

7.2 Slopes: Slopes in the vicinity of the dam are steep, approximately 50° to 55° since the dam is located at the bottom of a gorge.

7.3 Approximate Number of Homes and Population: There are no structures downstream until the Townes Dam is reached, approximately $3\frac{1}{2}$ miles downstream.

APPENDIX IV
GEOLOGY REPORTS

**GEOLOGICAL REPORT
ON THE
PINNACLES HYDRO-ELECTRIC DEVELOPMENT
ON THE DAN RIVER
IN PATRICK COUNTY, VIRGINIA
P.W.A. PROJECT 1151-R**

**FOR
THE CITY OF DANVILLE
DANVILLE, VIRGINIA**

JANUARY 1937

**BY
WILBUR A. NELSON
Consulting Geologist**

**SUPPLEMENT
TO ENGINEERING REPORT BY
CHAS. T. HAIN, INC
BOSTON, MASSACHUSETTS**

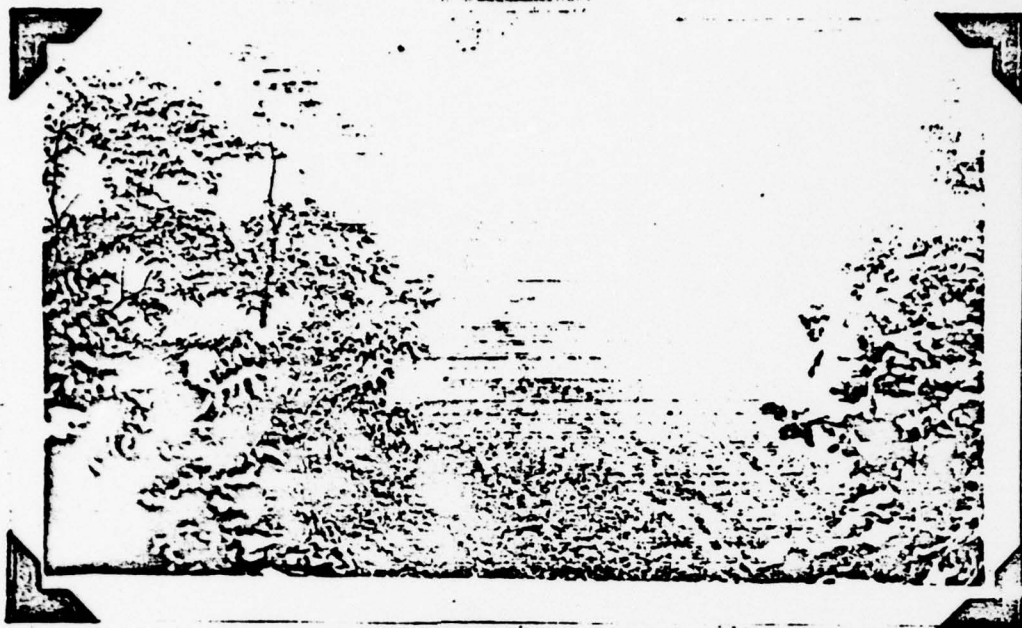
GEOLOGICAL REPORT
ON THE
PINNACLES HYDRO-ELECTRIC DEVELOPMENT
ON THE DAN RIVER
IN PATRICK COUNTY, VIRGINIA
P.W.A. PROJECT 1151-R

By
WILBUR A. NELSON
CONSULTING GEOLOGIST

INTRODUCTION

Field work in connection with this Report was begun in November, 1935, and was completed in November, 1936. From November, 1935 to July 1, 1936, several trips were made each month to the project. From July 1st to September 1st almost continuous work was done on this project.

Trips were made several times during each month to the different dam sites, which were studied and drilled during this period. In addition to a thorough checking of the cores of the drill holes, a detailed study was made of the geology of the dam site areas. Geological sections were obtained from surface outcroppings, from road



Pinnacles of the Dan

Photo by W.A.N.



Winter Scene in Gorge of Dan River at Point
Lookout Dam Site

Photp by W.A.N.

cuts and from the study of the core drilling done in this region. Considerable time was spent in studying the underground water conditions in the narrow ridge forming the east abutment of the Big Bend Dam Site.

After September 1, 1936 two trips were made to the project, one in September and one in November, and considerable office work has been done in the preparation of this Report.

This Report is made for the City of Danville, Danville, Virginia, and will be supplementary to the Engineering Report made by Charles T. Main, Inc., on this same Project for the City of Danville.

LOCATION

The Point Lookout Dam Site and the Big Bend Dam Site are located in the extreme west central part of Patrick County, Virginia, in the gorge of the Dan River near the eastern edge of the Blue Ridge Mountains.

The location of the Point Lookout Dam Site is approximately $9\frac{1}{2}$ miles slightly north of east of the town of Stuart, while the Big Bend Dam Site is approximately $7\frac{1}{2}$ miles slightly north of east of the town of Stuart.

The nearest railroad to the dam site is a Branch of the Southern Railway which stops at the town of Stuart.

Service roads have been built into both dam sites by the City of Danville. These roads connect with the main state highway at Cruzes Store on top of the Blue Ridge Mountains, about 5 miles roughly north of the two dam sites.

Patrick County is located in that part of Virginia known as the Blue Ridge Mountain Province, which borders on the east the Great Valley Region of Virginia.

The Point Lookout and Big Bend Dam Sites, and surrounding territory, are shown in the northeast quarter of the U. S. Geological Survey Topographic Sheet known as the "Stuart Quadrangle", a topographic sheet on the scale of 1 to 62500, having a contour interval of 20 feet. (See Plate 1).

GEOLOGY

The rocks comprising this part of Virginia are known as metamorphic rocks and belong to the great group of crystalline rocks designated as of pre-Cambrian Age.

All of the rocks outcropping and occurring around the two dam sites belong to this group of intensely metamorphosed and altered crystalline rocks. The rocks consist of a schistose series of gneisses of various compositions, some layers being graphitic, some sericitic, and some almost pure quartz. Most of the layers can be

designated a quartz mica gneiss, due to the large but varying amounts of mica and quartz comprising them. In addition smaller amounts of other rock-forming minerals occur in these layers. Interbedded in these series of quartz mica gneisses occur a number of layers of a rock known as greenstone, which is composed primarily of hornblend.

It is considered from an examination of thin sections of rock from the drill holes at both dam sites that the quartz mica gneisses of various compositions were originally sedimentary rocks, containing in some of the layers a small amount of calcium carbonate, but not sufficient to cause solution channels or danger from leakage due to a dissolving out of such calcium carbonate content. Interbedded with these altered sediments at the Big Bend Dam Site occur several layers of greenstone. These layers of greenstone, now composed almost entirely of hornblend, may have originally either been sills of basic igneous rock which came into this sedimentary series in pre-Cambrian Times, or may have been originally just other layers of the sedimentary series which contained a higher calcium content which, when subjected to the metamorphic forces to which this region has been subjected in the past, could have been altered to the present type of greenstone rock which occurs in this section.

Irrespective of the original origin of these greenstone layers, they are now dense layers of rock made up almost entirely of hornblende and lying parallel to the quartz mica gneisses of varying composition which occur between them.

All of the rock series at both dam sites show varying degrees of foliation and schistosity. Strike and dip observations taken on these rocks relate entirely to the planes of schistosity, which are prominently developed throughout this region. It is, however, thought that the schistosity developed at both dam sites is locally at least parallel to the old original bedding planes of these highly altered pre-Cambrian sedimentary rocks.

A study of these rocks shows at least three periods of folding, which can be distinctly recognized at the present time, but it is not known how many other periods of folding ante-dated the three that can now be distinguished. These distinct periods of folding have naturally fractured the rocks in many different ways and have developed planes of weakness along definite lines in certain places.

On the eastern front of the Blue Ridge, about five miles east of the dam site area, there is evidence of thrust faulting and also normal faulting, but as these lines of disturbances roughly parallel the front of the Blue Ridge, which has a direction of approximately N.60°E.,

these lines do not cut into dam site and reservoir areas.

At the Big Bend Dam Site, the service road built into the selected dam site, shows in one of the cuts made in September, 1936, the presence of a normal fault, which is almost vertical and approximately parallel to the top of the ridge just to the west of the Dan River at Dam Sites Nos. 21 and 22. Although this fault is only several hundred feet from the Dan River, its direction is such as not to affect adversely the dam sites just mentioned.

A detailed study of all of these complicated geological factors has been necessary in order to determine where proper foundation conditions exist for the location of dams at Point Lookout and in the Big Bend region.

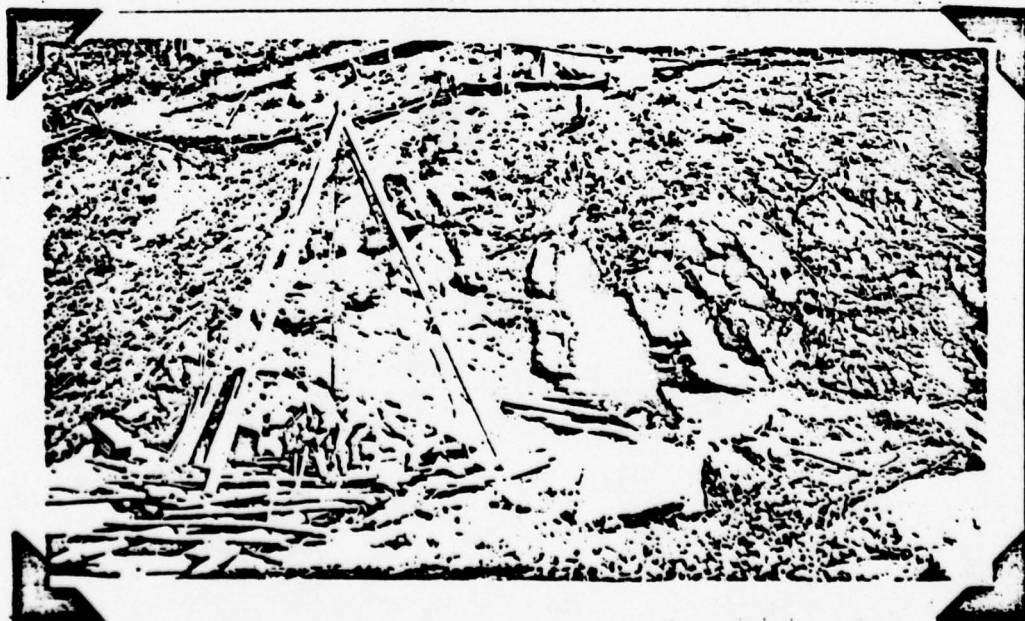
GEOLOGY AND STRUCTURE OF ROCKS
AT
POINT LOOKOUT DAM SITES

At the Point Lookout Dam Sites, the pre-Cambrian crystalline rocks consist of a series of quartz mica gneisses formed by metamorphism of pre-Cambrian sediments. Certain of these gneissic layers contain about three or four percent calcium carbonate, which in thin rock sections is shown now to be in the form of crystalline calcium carbonate acting as a cementing material in isolated and not in contiguous places.



Looking Downstream. Dam Site No. 16 in
the Foreground

Photo by W.A.N.



Drilling at Dam Site No. 16 on West Bank
of the Dan River

Photo by W.A.N.

An analysis of a large sample of rock from the service road to the Point Lookout Dam Sites, at a point about half way from the top to the bottom of the gorge on this road, shows that particular layer of rock to contain 3.72 percent calcium carbonate. This analysis was made of a sample of stone collected by the Consulting Geologist, and analysed by Freehling and Robertson, Inc., of Richmond, Virginia.

The original location at point Lookout was known as Dam Site No. 1. It was located near the center line of the rock slide from the Point Lookout bluff, and was condemned on geological grounds discussed further on in this Report.

The Consulting Geologist recommended a site to be drilled, which was originally designated as Dam Site No. 1. This site was downstream about 25 feet from the downstream side of the rock slide just mentioned. This is practically the same site as was designated later Dam Site No. 16 by the Engineers. Instead of drilling the site recommended by the Consulting Geologist a site, known as Dam Site No. 5, about 75 feet downstream from the junction of Round Meadow Creek and the Dan River and approximately 200 feet upstream from the rock slide area above-mentioned, was selected by the Engineers and drilling was started without the knowledge of the Consulting

Geologist, although this stretch of the river had been previously condemned by the Consulting Geologist. Details of the geological reasons for condemning this area are given later on in this Report.

Dam Site No. 17 is about 25 or 30 feet upstream from Dam Site No. 16, the site selected by the Designing Engineer. Dam Site No. 16, as already mentioned, is practically identical with the original Dam Site No. 11, the two sites being only 3 or 4 feet apart. Dam Site No. 16 is discussed after the "Character and Weathering of Schistosity" and before Dam Site No. 17, as the nearest drilling done to Dam Site No. 17 is on the location of Dam Site No. 16, and the information on which the location of Dam Site No. 17 is based is only the partial information obtained from drilling Dam Site No. 16.

Character and Weathering of Schistosity for the Point Lookout Dam Sites: For all of the Point Lookout Dam Site locations the general strike of the schistosity of the quartz mica gneiss in this region is from N. 45° E. to N. 55° E., and the dip of the schistosity varies from 50° to the South East to 80° to the South East. A cross section taken at right angles to the strike of the schistosity and showing the general dip of the schistosity

at the Point Lookout Dam Site No. 15 is attached. This cross section shows that the schistosity is steeper in the bottom of the gorge than it is on the sides of the gorge at the level of the top of the proposed dam.

A detailed study of the core drill holes drilled at Dam Site locations 11, 15 and 16, along this section, shows that lines of weathering, that is the lines along which the ground water has percolated downward and caused oxidation and weathering of the rocks, follows definitely the lines of schistosity and that these lines of weakness extend down on the west side of the Point Lookout Dam Site down to at least the level of the stream.

Placing Drill Holes P-A, 50 and 13 in this same plane, it can be shown that the line of weathered rock occurring in Drill Hole P-A at from 41'1 $\frac{1}{2}$ " to 41'6 $\frac{1}{2}$ " occurs again in the bottom of Drill Hole No. 50 at an elevation of 2150 feet above sea level, and again occurs in the bottom of Drill Hole 13 at an approximate elevation of 2121 feet where the core loss at this elevation amounts to 2 $\frac{1}{2}$ ". In the same way soft, weathered rock layers paralleling the schistosity are found to extend down to approximately this same depth by using data in Holes P-13, P-52, P-53 and P-53A, drilled for Dam Site locations 11 and 15. In all of the drill holes on the west abutment of the Point Lookout Dam locations 11, 15 and 16, soft, weathered core

rock is found within a few feet of the bottom of each hole. In Drill Hole No. 55, for Dam Site No. 15, a 45° hole drilled from near the center of the river under the western bank of the stream, we find iron stained joints at a depth of 48'1", or just 7 feet above the bottom of this hole, showing that at a depth of approximately 2075 feet we have rocks showing weathering and rocks through which leakage may occur.

Dam Site No. 16: Thin sections of the rock from the drill cores nearest to the proposed dam location at Point Lookout, which were drilled for Dam Site No. 16, show only very minor amounts of calcium carbonate occurring in the rocks at the dam site. Thin rock sections were made from an average part of the core from Drill Holes P-B, P-C, P-D, P-F, and P-J. Examinations of these thin sections were made under the microscope to determine the amount of calcium carbonate present in crystalline form. In Drill Hole P-B the rock, which was shown to be a quartz mica gneiss with definite banding of quartz and mica, showed a few crystals of calcite in one part of the thin section, as well as other rock-forming minerals in minor amounts. There was not enough calcite present to make the rock porous through the action of ground water in dissolving out the calcite.

Two thin rock sections were made from the core of Drill Hole P-C, one near the bottom of the hole and one about half way down. The thin rock section from the bottom of the hole showed no calcite, but the thin section made from the core about halfway down in this hole showed several calcite crystals acting as a cementing material between crystals of quartz. The rock in both of these thin sections was seen to be a quartz mica gneiss containing some garnets, and a number of other rock-forming minerals in minor amounts.

The thin rock section from the core of Drill Hole P-D showed this rock to be a quartz mica gneiss with one small aggregate of calcite as a cementing material between quartz crystals, as well as other rock-forming minerals in minor amounts. Again this microscopic investigation showed that there was not sufficient calcite present in the rock to cause rock porosity by the solution of this mineral.

The thin section from the core of Drill Hole P-F showed several small crystals of calcite with mica and quartz. The rock was seen to be a quartz mica gneiss, with some pyrite present and also a few garnets, as well as other rock-forming minerals in minor amounts. This thin rock section showed more calcite than was seen in any of the other thin sections made from the cores of the

drill holes at the Point Lookout Dam Site. However, the amount of calcite in this thin rock section was not sufficient to cause any trouble by solution through the action of circulating ground water in this type of rock as the several calcite crystals were all more or less segregated together in one part of the thin rock section.

The thin rock section made from the core of Drill Hole P-H did not show any calcite. The rock was again seen to be a quartz mica gneiss, with alternating layers of mica and quartz containing garnets and pyrite and also additional rock-forming minerals in minor amounts.

The thin rock section made from a sample of the core of Drill Hole P-J showed one or two small calcite crystals in the mixture of quartz and mica. This thin rock section also showed some pyrite, as well as additional rock-forming minerals in minor amounts. The rock was seen to be a quartz mica gneiss.

In all of the thin rock sections examined and mentioned above, the calcite in no case was found to be scattered throughout the entire thin rock section, but, wherever it was found, the calcite occurred only in isolated spots in the rock and then only in minor amounts in crystalline form with the quartz and mica and other rock-forming minerals. In only one drill hole in the Point

Lookout area, which was one of the holes drilled for Dam Site location No. 15, was any calcite seen in sufficiently large crystal form to be recognized by the eye. In Drill Hole P-60, at 21' depth, one calcite crystal at least $\frac{1}{2}$ inch across was seen in the drill core.

Dam Site No. 17: The final location of the Point Lookout Dam, Dam Site No. 17, made by the Designing Engineer, has moved the dam location upstream approximately 30 feet from Dam Site No. 16, the area in which the final drilling was done, and has placed the west side of this dam on the west side of the gorge in the edge of the rock slide from the high Point Lookout bluff.

Although core drilling has been done on the downstream side of this final location none has been done on the upstream side, except Hole P-K for abutment purposes, so that no definite information is available as to the thickness of the rock slide on the west side of the gorge where the dam is now located. As this location is on the edge of the rock slide, it is logical to assume that the slide has a less thickness than it would have near its center part, another 90 to 100 feet upstream.

As each rock slide from a bluff acts in a slightly different manner, there is no way of definitely determining the amount of rock debris lying on the west side of the

gorge at the present dam site location unless additional drilling is done at this point, and any subsurface contours made to solid rock at this point are purely hypothetical. The same is true of any contour lines drawn on the top of unweathered rock on the east side of the present dam location, as no drill holes have been dug except on the downstream side of this location, the nearest ones being about 30 feet away from the center line of the proposed location. As the schistosity of the rock on this side cuts across the dam location at about an angle of 45° we have again no way of determining the depth to solid rock except by deductions. Therefore, no maps showing contour elevations of solid rock are given in this Reprt.

At Dam Site No. 16, soil or loose rock covering varies from 14'4" at Hole P-K, approximately 30 feet upstream from the edge of the rock slide, to 6 feet along the hillside line of the dam site on the west side of the gorge, while on the river banks the soil overburden is about 4 feet. On the east side of the gorge the overburden to weathered rock varies from 4'8" to 7'1" along the line of the dam site. So it is logical to assume that at Site No. 17 one will find at least this much overburden and probably more.

The depth from the surface to the unweathered rock on the hillside line of Dam Site No. 16, on the west side of the gorge, varies from 26'3" to 35'7", but in Hole P-K, an abutment hole on the west side of the gorge, the depth to good rock is 44'3½". Taking Holes A, C, and K, which in order named are located toward the edge of the rock slide, it is noted that the depth to good rock increases, being 34'4" in Hole P-Q, 35'7" in Hole P-G, and 44'3½" in Hole P-K. This indicates that it will be approximately 40 feet to unweathered rock at the west end of Dam Site No. 17.

If the depth to unweathered rock increases regularly upstream from each of the other holes drilled on the line of Dam Site No. 16, it might be estimated that the depth to unweathered rock along the arc of Dam Site No. 17 on the west side of the gorge would vary from 30 to 35 feet, plus any additional depth due to the edge of the rock slide. These facts are given as nothing more than an estimate based on the facts given above.

On the east side of the gorge the depth to good rock varies from 16'2½" to 25'9", these data being obtained from three holes drilled on the line of Dam Site No. 16. A study of the surface topography on this side of the gorge would indicate that the depth to weathered rock on the line

of Dam Site No. 17 would be at least as deep as shown at Site No. 16, and there is always the possibility that it might be a few feet deeper.

In order to take care of any leakage which may occur under the Point Lookout Dam Site, selected by the Designing Engineer, on its west side and under the river part of the dam, grout holes should be put down to at least an elevation of 2,070 feet above sea level as the strike of the schistosity does not parallel the dam but extends across the line of the dam at an angle of approximately 45° , which would permit of leakage from the reservoir along such lines under the dam into the river below. (See page 10).

From a geological standpoint, it is considered that a site about 30 to 50 feet downstream from the final location is better than the location decided upon by the Designing Engineer, unless additional drilling had been done to prove all of the facts in regard to this final location to be definitely satisfactory. A location 30 to 50 feet downstream would have kept entirely away from the edges of the rock slide on the west side of the gorge, and one would be dealing with definitely known rock conditions.

Other Dam Sites Investigated at Point Lookout: The original dam site at Point Lookout, Dam Site No. 1, was located in the narrowest part of the gorge, which happened to be in the center of the rock slide from Point Lookout, this heavy slide of loose rock having filled in the gorge and made the stream narrowest at this point. Detailed geological work showed, from a study of the strike and dip measurements of the outcrops of rock on the west side of the gorge, that none of these rock outcrops were in place, as the schistosity of these rock outcrops when plotted on the map showed a fan-shaped pattern. Although the geological investigation showed definitely the presence of this rock slide, the Engineers wished to drill several holes at this point in order to prove the correctness of the geological facts, and, therefore, Drill Holes P-1, 2 and 3 were drilled in the rock slide area and showed that the geological facts as stated by the Consulting Geologist were correct.

Dam Site No. 5: The site recommended by the Consulting Geologist as the one then to be drilled was designated as Dam Site No. 11 and was about 25 feet downstream from the rock slide. This is practically the same site as was later given the Number 16 by the Engineers. Instead of

drilling this site, a new site, known as Dam Site No. 5, about 75 feet downstream from the junction of Round Meadow Creek and the Dan River, and approximately 200 feet upstream from the rock slide area first drilled, was selected by the Engineers and drilling was started without the knowledge of the Consulting Geologist, although previously this stretch of the river had been condemned by the Consulting Geologist, on geological grounds as being unsuitable for the location of the Point Lookout Dam. Nineteen holes were drilled on this location, none of which gave information of any geological value, as all of this drilling resulted in only condemning a site which, as stated above, had been condemned by the Consulting Geologist. All of these holes were unnecessary and none of them should have been drilled.

At this particular site, the schistosity of the rocks paralleled the thread of the Dan River, and just upstream from this site the Dan River turned sharply to the southeast at the point where Round Meadow Creek came in from a direction slightly west of north, so that the schistosity of the rocks on the east side of the gorge cut across the Dan River above and below the proposed location and made leakage through the abutment of a proposed dam at this point too dangerous and made the abutment conditions too bad to consider this site. On the

west side, the schistosity likewise cut through back of the abutment from Round Meadow Creek above this proposed site into the rock slide on the west side of the gorge below this site, again making leakage from the reservoir through the west abutment into the stream below this proposed site a strong possibility. All of these conditions were readily worked out by the detailed geological investigations made by the Consulting Geologist and were the reasons for condemning this site without drilling.

Summary of all Sites Investigated at Point Lookout: When the geological studies of this region were completed it was determined that the only satisfactory location, from a foundation standpoint, for the dam site was in that section of the river below Dam Site Location 14 and above Dam Site Location 15, a distance of approximately 90 feet and this fact was reported to the proper officials of the City of Danville and to Charles T. Main, Inc.

In the section of the river between Dam Site No. 15 and Dam Site No. 17, the strike of the schistosity of the rock which varies from N. 45° E., to N. 50° E., crosses the thread of the river at an oblique angle, so that any leakage along lines of weakness paralleling the planes of schistosity will be much more easily stopped

than where the schistosity parallels the thread of the river above the rock slide, the lower edge of which is at Dam Site No. 17. In addition to this fact the rock outcrops are more massive and more continuous on both sides of the river in the section selected than in the sections above and below. Within this section, between Dam Site Locations 15 to 17, a distance of approximately 30 feet, it is desirable, as already mentioned, to stay slightly downstream from Dam Site No. 17, so as to get away from the edge of the rock slide of the high bluff. Recommendations consistently have been made throughout the progress of this work that a site slightly downstream from 17 and upstream from 15 be selected as the best site from a geological standpoint.

Dam Sites 11, 15, 16 and 17 are all located very close together, Dam Site No. 15 being approximately 50 feet downstream from Dam Site No. 18, and Dam Site No. 16 being approximately 25 feet downstream from Dam Site No. 17. Dam Sites 11 and 18 are at almost the same or identical locations, as they are located only 3 or 4 feet apart. The drilling in connection with these four dam sites was started at first for Dam Site No. 11. Due to the fact that the Designing Engineer did not consider that the western abutment was satisfactory, drilling was

moved downstream to Dam Site No. 15, then upstream to Dam Site No. 16, and Dam Site No. 17 was finally located approximately 25 feet upstream from the drilling done for Dam Site No. 16.

RECOMMENDATIONS

Recommendations on the Point Lookout Dam Site, as well as the Big Bend Dam Site, are given at the end of the entire Report.

GEOLOGY AND STRUCTURE OF ROCKS

AT BIG BEND DAM SITES

Original Big Bend Dam Site at Big Bend (Site A): At the original Big Bend Dam Site (Site A) the pre-Cambrian rocks consist of a series of quartz mica gneisses. At this point the general strike of the schistosity of these gneisses is from N. 45° E., to N. 60° E., and the dip of the schistosity, which is to the southeast, varies from 60° to 80°.

The original Big Bend Dam Site was located on the Dan River just below the mouth of Ivy Creek. The first geological investigations were made at this point and showed that the west abutment of the dam was against a long narrow ridge formed by the Big Bend of the Dan River, while the east abutment of the dam was against a narrow ridge lying between the Dan River and Ivy Creek. Both of the abutments to this proposed dam were on narrow ridges where leakage conditions are undoubtedly bad. The schistosity of the gneissic rocks was at right angles to both of these ridges, so that the lines of weathering along the planes of schistosity extend across the narrow point of both of these ridges and would undoubtedly permit leakage from the reservoir into the Dan River below the proposed dam site. This would have called for excessive grouting on both of

these narrow ridges, with also the possibility that such grouting would not have stopped the desired amount of leakage. These detailed studies of the geology at this original site showed the geological conditions were so unfavorable for the location of a dam the height desired by the Engineers that this site was condemned and abandoned.

Dam Sites 1 to 8, All Downstream from Big Bend: Following the request of the Engineers that the first site below this point on the Dan River where geological conditions were satisfactory would be the point where they would prefer to locate a dam, a study was started of the upper Dan River Gorge. Sites from 1 to 8 were investigated (see attached map for locations). Cross sections of the gorge made by the Engineers showed that the stretch of the river at Sites 5 and 6 were narrowest. Detailed geological investigations were made at and between these points, which were only a few hundred feet apart. The final locations, known as Dam Sites 21 and 22, are approximately at the location of original Site No. 5.

Dam Sites Nos. 5 and 6, Approximately 2 Miles Downstream from Original Big Bend Site: In the stretch of the river at original locations 5 and 6, a study of the detailed geology shows that the crystalline pre-Cambrian rocks consist of a series of quartz mica gneisses of varying composition

interbedded with beds of greenstone composed mostly of hornblende. The strike of the schistosity varies from N. 45° E., to N. 60° E., and the dip of the schistosity varies from 50° to 85° to the southeast. The strike of the schistosity extends across the Dan River at almost right angles, and the dip of the schistosity, which is in general almost vertical, is downstream.

Dam Site No. 1 (new series of numbers) Approximately Same as Dam Site No. 6: The best site in this stretch of the river from a geological standpoint is what is shown as Site No. 1 on the attached map showing the general geology of this area. This site is, from a geological standpoint, the best of the sites studied, as both the east and west abutments of this proposed dam are against the sides of the gorge where the bends of the river do not make narrow ridges and where no leakage would occur around their abutment. Due to the cross section at this point being greater than at the other points studied the Engineers considered it was necessary to attempt to find a narrower cross section where geological conditions were satisfactory, or where a study of these conditions would show that any weakness in the foundation or abutments could be taken care of by proper engineering practice.

The narrowest cross section on this section of the river was upstream several hundred feet from Site No. 1,

and to this narrowest cross section the designations Dam Sites Nos. 21 and 22 were given. Between these two sites, 1 to 22, a distance of approximately 550 feet, several drill holes were put down on the west side of the gorge at the then proposed reservoir height to see if satisfactory cores could be obtained which would show the presence of a type of unweathered rock which would suit the Designing Engineer for abutment purposes. Although two of these holes showed considerable good rock in the drill cores and there was also a 15 foot exposed ledge of hard quartz mica gneiss, it was not considered by the Designing Engineer that sufficient good rock was present in this stretch of the river for satisfactory abutments. Since then the blasting done for the service road into the dam site has shown for a distance of 250 feet downstream from the west abutment of Dam Site No. 22 quartz mica gneiss which is considered satisfactory for abutment purposes by the Consulting Geologist.

Dam Sites Nos. 21 and 22 (new series of numbers) Approximately same as Dam Site No. 5: Drilling was then concentrated at Dam locations 21 and 22 (the arcs of these two locations cross on the east bank of the river) although the east abutment of both of these dam sites is against a narrow ridge made by the bend of the Dan River where geological conditions are most unfavorable and where leakage

will undoubtedly occur, which will have to be stopped by proper engineering practice.

Drilling at Dam Sites Nos. 21 and 22 showed satisfactory foundation conditions to exist along the arc of either of these dam sites, and satisfactory abutment conditions to exist at the west abutment of both sites.

Final drill holes put down to determine the foundation conditions for Dam Site No. 22 showed overburden to vary from two feet to sixteen feet, and the top of unweathered rock to vary from six feet below the surface to 54 feet below the surface, the greater depths to the unweathered rock occurring on the upper portion of the arc on the west side of the river, where, although the rock is deeply weathered, abutment conditions are excellent.

The entire west bank of the river between Dam Site No. 1 and Dam Site No. 22 is more deeply weathered than on the east side, and depths to unweathered rock are much greater than on the east side of the gorge. Below the first unweathered rock there occur weathered schistose rock in narrow planes parallel to the schistosity, which zones will continue below the dam foundations and should be grouted if excessive leakage is to be prevented.

However, the geological conditions along the arc of Sites Nos. 21 and 22 and the character of rocks along these arcs are considered satisfactory for dam foundation purposes.

Dam Site No. 22: Thin rock sections were made of cores of holes drilled along the arc or in close proximity to the arc of Dam Site No. 22. The thin rock section made from part of the core from Drill Hole B-222 showed the rock to be a quartz mica gneiss containing much pyrite as an impurity, and also other rock-forming minerals. No calcite was present.

The thin rock section from part of the core of Drill Hole No. 223 was in what is known as greenstone and showed that this rock was made up almost entirely of thin crystalline hornblend^e, with accessory rock-forming minerals.

The thin rock section of part of the core of Drill Hole No. B-226 was also in greenstone and showed that this rock was of the same character as rock in Drill Hole No. B-223.

The thin rock section from part of the core of Drill Hole No. B-224 shows this rock to be primarily a mica schist, full of small drag folds and having considerable pyrite following these small folds. No calcite is present but quartz and garnets do occur, as well as other rock-forming minerals.

The thin rock section from part of the core of Drill Hole No. B-230 showed this rock to be a quartz mica gneiss, banded with graphite gneiss. It contains much

so small that it would not affect the solubility of these rocks when considered from the standpoint of dam foundations.

The schistosity of these crystalline rocks appears to be roughly parallel to the original bedding of these rocks. These rocks, with the exception of the greenstone, are of a type that would weather more deeply than the quartz mica gneisses found at the Point Lookout Site which contain, as a rule, more quartz although there are one or two 10 or 15 foot layers of the quartz mica gneiss at the sites being discussed which contain much quartz and are very massive and resistant to weathering. They show up beautifully in the cuts of the service road which was completed in September, 1936, to the Big Bend Site.

Dam Site No. 22, which it is understood is the final selection of the Designing Engineer for the location of the so-called Big Bend Dam, lies entirely in quartz mica gneiss of varying composition. The west abutment of this dam is in quartz mica gneiss and approximately 50 feet from the upstream edge of a massive greenstone layer, while the east abutment is approximately 25 feet from the upstream edge of this same massive greenstone layer.

These statements are based on the location of the dam as shown on Exhibit C, Sheet 1 of the Pinnacles Hydro-Electric Project, with a reservoir level elevation of 2532 feet.

AD-A063 581

ARMY ENGINEERING DISTRICT NORFOLK VA
NATIONAL DAM SAFETY PROGRAM. TALBOTT DAM NUMBER (VA 14101), PAT--ETC(U)
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Detailed Studies of Narrow Ridge on which East Abutment

of Dam Site No. 22 is Located: The location of the east abutment of the dam is of considerable importance, as this abutment is located on a long, narrow ridge through which leakage will undoubtedly occur from the reservoir formed by the dam into the river below unless this leakage is taken care of by proper engineering practice. Between this east abutment and the top of the narrow ridge which occurs on this side of the river at Drill Hole No. 302 there are several hundred feet of rock through which leakage will occur, as the water table in this ridge between the abutment and to within a few feet of Drill Hole No. 302 is below the elevation of the proposed reservoir water level.

Drill Hole No. 301, put down on the top of the ridge near the east abutment and in the greenstone above-referred to, had a bottom elevation of approximately 2510 feet and no water remained in this hole although it was tested continuously throughout the entire summer and fall of 1936 to see if it contained water at any time. This greenstone is a dense rock, fractured by jointing produced through the folding forces to which all of these rocks have been subjected.

A cross section of the ridge showing this hole and eight other drill holes which were put down along the crest of this ridge is attached, which shows the level of



Gorge of Dan River
Photo by W.A.N.



Narrow Ridge forming East
Abutment of Dam Sites 21
and 22

Photo by W.A.N.

the water table on July 28, 1936, on August 17, 1936, and on September 21, 1936, which later date was the date on which the lowest water table elevation was reached, for in October the water table started to rise slightly.

In addition to the water table, the attached cross section shows the depth to hard rock and the depth of soil overburden.

Nine drill holes were put down along the top of this ridge as it was known that the geological conditions along this ridge at the east abutment of the proposed dam were very poor. It should be noted that the water table line, given on July 28, 1936, was only between Holes No. 303 and 302. From Drill Holes No. 302 to 301 no data was available on the top of the water table, as on that date the top of the water table was below the bottom of Hole No. 301 and below elevation 2522 in Hole No. 309, which was as deep as any tests could be made in that hole on that date as the lower two feet of this hole had caved in. The reservoir water table is given on the attached cross section at 2532 feet above sea level. If this is the elevation used when the dam is constructed the water table of July 28 would be above this elevation, except between Drill Holes No. 302 and the east abutment of the dam, a distance of about 240 feet. The lowest water table obtained to date was on September 21, 1936,

at which time this water table parallels the water table of July 28 between Drill Holes No. 302 and 306, being approximately 6 to 10 feet below the July 28 level. No water table level is given on the September date for Hole No. 300A, which lies between Drill Holes Nos. 306 and 308, due to the fact that this hole evidently caved at a depth of 77 feet below the surface, or at an elevation of 2585 feet, but in Drill Hole No. 308 the September 21 water level is approximately 8 feet lower than the July 28 level. As Hole No. 300-A caved at an elevation of 2585, the water table in this hole on the September date was below this elevation, and it is estimated it was on the September date at approximately 2578 feet elevation or 7 feet below the point at which the hole caved. The information on this hole caving was obtained from Mr. D. J. Shea, Resident Engineer, in letter dated January 16, 1937.

Attached are the observations made under the direction of the Engineers at the request of the Consulting Geologist on the water level in the drill holes on the ridge at the east abutment of the dam. It will be noted that throughout the entire time that observations were made no water level occurred in Holes Nos. 301 and 309, and that from August 1 to August 5 no observations were made in Hole No. 302, and that after August 17 no water was found in Holes Nos. 300A or 307. The 8 foot drop of water

ELEVATION OF WATER IN RIDGE HOLES

Month of July 1936

HUB ELEV. HOLE NO. DATE	2662.0 300A	2613.3 301	2620.3 302	2650.0 306	TIME	REMARKS
21	2606.1	NoWater	2559.0	2566.5	8:30 A.M.	(Shower from 11:
22	2606.1	" "	2558.8	2566.5	8:30 A.M.	(to 1:00 -21
23	2586.2	" "	2558.8	2566.4	9:00 A.M.	(Hard rain 4:30-
24	2586.2	" "	2558.8	2566.5	9:00 A.M.	(5:30 P.M. -23rd.
25	2586.5	" "	2558.8	2566.5	8:30 A.M.	
27	2586.4	" "	2558.5	2566.5	8:30 A.M.	(40' -1 & 60' of (1 1/2" put in 302
28	2586.3	" "	2558.8	2566.5	9:00 A.M.	(Hard Rain 2:00-
30	2585.9	" "	2558.8	2566.4	9:00 A.M.	(4:00 P.M. -29th
31	2585.9	" "	2558.8	2566.5	9:00 A.M.	

Month of August 1936

1	2585.7	" "	2558.0	2566.0	9:00 A.M.	
3	2585.7	" "		2566.0	8:45 A.M.	(Tester would not (go by 53.5 in X
4	2585.5	" "		2566.0	9:00 A.M.	Ditto
5	2585.4	" "		2566.0	9:00 A.M.	(Hard rain 11:45- (12:45 P.M.
6	2585.3	" "	2560.2	2566.0	10:00 A.M.	Rain 3 to 4 P.M.
7	2585.2	" "	2561.4	2565.8	9:00 A.M.	Rain 3 to 7 P.M.
8	2585.2	" "		2566.0	8:30 A.M.	to 10.00 A.M.
10	2585.0	" "	2562.0	2565.7	9:45 A.M.	Rain 12:30-5 P.M.
11	2584.8	" "	2562.1	2565.6	9:45 A.M.	
12	2584.8	" "	2562.0	2565.5	9:15 A.M.	
13	NoWater	" "	2561.9	2565.5	1:00 P.M.	
14	2584.6	" "	2561.7	2565.5	9:00 A.M.	
15	2584.6	" "	2561.5	2563.4	9:15 A.M.	(Rain, slow 3:30 (to 6:00 P.M.
17	2584.6	" "	2562.0	2565.2	8:45 A.M.	(Rain 8-10 P.M. (Aug. 16.
18	NoWater	" "	2561.9	2565.2	9:00 A.M.	
21	" "	" "	2561.4	2564.9	9:00 A.M.	
31	" "	" "	2560.8	2564.4	8:45 A.M.	

Month of September 1936

HUB ELEV. HOLE NO. DATE	2662.0 300A	2613.3 301	2620.3 302	2650.0 306	TIME	REMARKS
1	NoWater	NoWater	2560.2	2564.4	2:00 P.M.	
2	" "	" "	2560.2	2564.3	8:45 A.M.	
7	" "	" "	2560.0	2564.1	8:45 A.M.	
14	" "	" "	2559.1	2563.6	9:00 A.M.	
21	" "	" "	2558.5	2563.2	9:45 A.M.	

Month of October 1936

8	" "	" "	2559.9	2562.7	8:30 A.M.	(Heavy Rains Sept. 29 & 30.
12	" "	" "	2559.8	2562.2	8:30 A.M.	(Rain 7, 8 & 9.
19	" "	" "	2563.1	2562.0	8:30 A.M.	(Heavy Rain 16th Light Rain 17th
26	" "	" "	2561.9	2561.7	8:45 A.M.	

ELEVATION OF WATER IN RIDGE HOLES (CONT'D)

Month of July 1936

HUB ELEV. HOLE NO. DATE	2638.0 307	2654.7 308	2616.0 309	TIME	REMARKS
21	2537.0	2573.1	No Water	8:30 A.M.	309 Caved at 2520.
22	2536.9	2572.9	" "	8:30 A.M.	309 Caved at 2520. (Shower from 11.45- 1:00 on 21st.
23	2536.8	2572.9	" "	9:00 A.M.	309 Caved at 2522.
24	2536.9	2573.1	" "	9:00 A.M.	309 Caved at 2522. (Hard Rain 2 - 4-23
25	2536.8	2572.4	" "	8:30 A.M.	
27	2536.7	2572.4	" "	8:30 A.M.	
28	2536.7	2572.4	" "	9:00 A.M.	(Hard Rain 2 to 4 a July 29.
30	2536.6	2571.9	" "	9:00 A.M.	
31	2536.7	2572.2	" "	9:00 A.M.	

Month of August 1936

1	2536.5	2571.9	" "	9:00 A.M.	
3	2536.0	2571.9	" "	8:45 A.M.	{Tester wou
4	2536.0	2571.8	" "	9:00 A.M.	
5	2535.9	2571.8	" "	9:00 A.M.	(Hard Rain 11:45 to 12.45 P.M.
6	2535.8	2571.8	" "	10:00 A.M.	Rain 3-4 P.M.
7	2535.8	2571.2	" "	9:00 A.M.	(Slow Rain 3 to 7 P
8	2535.7	2571.4	" "	8:30 A.M.	(to 10:00 A.M.-8th
10	2535.7	2571.4	" "	9:45 A.M.	Rain 12:30-5:00 P.M
11	2535.0	2571.1	" "	9:45 A.M.	
12	2534.9	2571.1	" "	9:15 A.M.	
13	2535.1	2571.0	" "	1:00 P.M.	
14	2535.1	2571.1	" "	9:00 A.M.	
15	2534.8	2570.8	" "	9:15 A.M.	(Rain, slow 3:30- 6:00 P.M.
17	2534.7	2570.4	" "	8:45 A.M.	Rain 8-10 P.M. 16t
18	Water Past Tape	2570.5	" "	9:00 A.M.	
21	Below Tester	2569.8	" "	9:00 A.M.	
31	No Water	2567.8	" "	8:45 A.M.	

Month of September 1936

HUB ELEV. HOLE NO. DATE	2635.0 307	2654.7 308	2616.0 309	TIME	REMARKS
1	No Water	2567.4	No Water	2:00 P.M.	
2	" "	2567.5	" "	8:45 A.M.	
7	" "	2566.5	" "	8:45 A.M.	
14	" "	2565.4	" "	9:00 A.M.	
21	" "	2565.0	" "	9:45 A.M.	

Month of October 1936

2	" "	2564.6	" "	8:30 A.M.	(Heavy Rains Sept. 29 & 30.
12	" "	2563.7	" "	8:30 A.M.	Rain 7, 8 and 9.
19	" "	2564.3	" "	8:30 A.M.	Heavy rain Oct. 16.
26	" "	2563.5	" "	8:45 A.M.	Light rain Oct. 17.

which was the average lowering in the water table from July 28 to September 21 would carry the water table to just below the bottom of Hole No. 307, and would carry it below the elevation of 2532, which it is understood is the proposed water level of the reservoir. This would mean that special attention should also be paid to stopping reservoir leakage around Drill Hole No. 307.

It should be noted that a particularly soft layer of gneiss containing a certain amount of sericite occurring at Drill Hole 302 is responsible for the deep overburden at this point, and that the layer of greenstone starting at Drill Hole No. 309 and extending down the ridge approximately 115 feet shows very little weathering compared to the weathering of the gneissic layers. It should also be noted that the greenstone occurring at this point, where the ridge is very narrow, is badly fractured and that the water table drops to an unknown depth in this rock. It is a type of rock which can be grouted easily.

It should also be noted that where a similar greenstone ledge occurs where the ridge is wide at Drill Hole No. 308 that the water table is found to occur at a satisfactory level.

The fact that the schistosity of all of these rock layers extends at right angles across the narrow part of the ridge makes a bad geological condition, as leakage will

occur along the weathered planes of schistosity which cut across the narrow part of the ridge at almost a right angle.

The results of the core drilling of this ridge shows that between Drill Hole No. 302 and the east abutment of the proposed dam site there is a stretch of rock partly greenstone and partly gneiss of varying composition through which leakage will occur, but it is considered that this leakage can be stopped by proper engineering practice. This is a distance of approximately 240 feet. Throughout most of the remainder of this distance the top of the water table, which is only based on records kept for several months since these drill holes were completed, is above the top of the proposed reservoir to be formed by the building of the proposed dam.

Normal Fault in Big Bend Dam Site Area: A normal fault occurs on the west side of the ridge which forms the west abutment to Dam Sites Nos. 21 and 22. It is exposed at Station 220-50 on the service road into this dam site area, where this normal fault cuts off part of the greenstone layer which crosses the river a few feet down stream from Dam Sites Nos. 21 and 22. This fault extends along the west side of the ridge in a direction N. 8° W., so it does not cut into the reservoir area or the Dam Sites Nos. 21 and 22.

RECOMMENDATIONS

Point Lookout Dam Site No. 17

From a geological standpoint Dam Site No. 17, the final location made by the Designing Engineer, is approved, with the understanding that, due to the fact that no core drilling has been done on the upstream side of this dam site and, further, due to the fact that the west abutment and part of the west side of the arc of this dam is located on the edge of the rock slide from Point Lookout, it is impossible to determine accurately the depth to solid rock. No sub-surface contours to solid rock could be drawn at this point as such would be purely hypothetical. It is estimated that at least 35 feet of overburden and weathered rock will be found on part of the west side of the arc of Dam No. 17.

It is also estimated from geological conditions that lines of weathering parallel to the planes of schistosity will extend down to an elevation of 2070 feet above sea level at places on the west side of the arc of this dam, so that grouting should be done to at least this depth.

Although from a geological standpoint Dam Site No. 16 is considered best, the site selected by the

Designing Engineer, Dam Site No. 17, is approved from a geological standpoint with the understanding that considerable grouting will have to be done and that considerable overburden will be found on part of the west arc of this dam site.

Big Bend Dam Site No. 22

Either Dam Site No. 21 or 22 is approved from a geological standpoint with the understanding that the provisions which are outlined below will be followed by the Engineers.

After the dam is built at or approximately at location No. 21 or 22 and after the leakage which will occur between the east abutment and Drill Hole No. 302, a distance of approximately 240 feet, has been taken care of by proper engineering practice, it is recommended that the reservoir be filled so that it can be determined just where any minor leakage may occur through the remainder of the ridge on the east side of the dam back of Drill Hole No. 302, and when the location of such leakage is determined that the water level of the reservoir be lowered and that this leakage be stopped by grouting that part of the ridge where the leakage occurred. Data obtained through the summer and early fall of 1936 on the

water table on this ridge shows that this water table is practically at the elevation of the top of the proposed reservoir level or above it throughout the remainder of the ridge, if the reservoir level is at 2532 feet above sea level. At Drill Hole No. 307 the water table was below the proposed reservoir level during the latter part of August, and the months of September and October. Leakage can, therefore, be expected through the ridge at this point, which leakage can be stopped in the manner suggested above.

Although from a geological standpoint it is considered that the site between 200 and 250 feet downstream from sites Nos. 21 and 22 is best, the sites selected by the Designing Engineer, that is Sites Nos. 21 and 22, are approved from a geological standpoint provided the recommendations made above are followed.

Respectfully submitted,

Wilbur A. Nelson
Wilbur A. Nelson,
Consulting Geologist.

January, 1937.

APPENDIX V

UNDERWATER INSPECTION REPORT



LOGAN DIVING, INC.

5731 ST. AUGUSTINE ROAD - CODE 904 731-0000
JACKSONVILLE, FLORIDA 32207

UNDERWATER INSPECTION

TALBOTT & TOWNES DAMS

FOR

CITY OF DANVILLE, VA.

APRIL 4 & 5, 1978

An underwater inspection of the Talbott and Townes Dams, also known as the Pinnacles Hydro-Electric Development, was requested and performed as part of a periodic general preventive maintenance program instituted by the Danville Electric Department.

The work was coordinated by Messrs. Eldred C. Yerks, H. E. Cole, and Clyne Willis of the Danville Electric Department. Their efforts included scheduling of personnel and materials to effect repairs anticipated to a 6" by-pass valve at the Talbott Dam and replacement of a lifting cable for the trash rack at Townes Dam; provide personnel to operate various valves and winches at both dams; coordinate water flow outages; provide safety liaison with Logan Diving, Inc.; and general coordination of the overall project.

Logan Diving, Inc. was represented by Jack S. Mixer, John Redon and Thomas E. Thurston. Their responsibilities were to inspect the upstream intake structures and appurtenances and downstream dam toes and footings.

The following procedure was followed at both dams prior to commencement of diving operations:

- 1.) Conference with the Danville project coordinators to detail inspection procedure, safety measures and exchange of other information.
- 2.) Physical operation of all gates and valves by Danville personnel in the presence of Logan representative to assure their good operation and to determine the extent of any leakage.
- 3.) Inspection of the downstream side of the dam by Logan for undermining, spalling concrete, leakage and other deleterious conditions.
- 4.) Groundline soundings and measurements by Logan on the upstream side of the dams in the vicinity of the intake.
- 5.) Set up and testing of diving equipment before each dive and after all flow valves had been closed.

TALBOTT DAM

DOWNSTREAM AREAS INSPECTED

Footings, stilling basin, drain holes in stilling basin wall, construction joints, discharge pipes, concrete deterioration.

DOWNSTREAM FINDINGS

- A.) Two stilling basin wall drain holes were clogged with gravel and debris. Both drains were opened during the inspection.
- B.) Two of the 6" ϕ discharge pipes had blockages which prevented totally free water flow. These blockages were not removed though minor attempts were made. This problem was not deemed significant enough to warrant extensive effort for removal and has existed for many years.

UPSTREAM AREAS INSPECTION

Gate guides, gate cable, groundline build-up concrete deterioration, trashrack, trashrack guides and supports.

UPSTREAM FINDINGS

In all regards the upstream portion was found to be in good condition with the one exception of extensive groundline build-up in front of the trash rack which is restricting water flow and placing pressure against the trash rack when the gate is open. (See drawing for details)

The groundline materials are a composite of silt, trash, sticks and tree stumps accumulated since the time of construction.

At the conclusion of the inspection, the gate was opened and the cable clearly marked to indicate the open position.

TOWNES DAM

DOWNSTREAM AREAS INSPECTED

Footings, spillway apron footings for undermining, construction joints, concrete deterioration and general condition.

DOWNSTREAM FINDINGS

No defects were found.

UPSTREAM AREAS INSPECTED

Trash rack guides, trash rack, groundline build-up concrete deterioration. Specific attention was given to the deteriorated and broken lifting cable, single wheel block and trash rack lifting eye.

UPSTREAM FINDINGS

No significant defects were found with the single exception of the broken lifting cable and corroded block.

Divers removed the old block and cable and attached the new block and cable furnished by Danville.

The new block was placed through the trash rack lifting eye and wired in position with approximate No. 9 copper wire.

Tension on the new stainless steel was pulled to remove all slack and the cable clearly marked in the closed trash rack position at the operating floor with red paint.

Rust blisters were observed on the trash rack guides which poses no structural problem at this time. However, diver should scrape and clean the guides before lifting of the trash rack to prevent any binding of the rack in the guides. (see drawing for groundline details)

RECOMMENDATIONS FOR TALBOTT DAM

The most significant problem is the heavy amount of silt and debris accumulated against the trash racks. Although this condition presents no imminent danger because of the structures good condition, it does restrict water flow and place undue stress on the trash racks supports.

Should this groundline build-up cause the trash rack to fail, the corrective cost, possible long term outage, and associated problems would be excessive.

We strongly recommend excavation of the groundline in front of the trash rack to allow full flow of water through the trash rack.

This work would require careful coordination with plant operations, careful excavation to avoid snagging the trash rack or structure which could cause damage and divers to monitor the work.

Our firm is designing a procedure and quotation for performance of this work.

Although the two clogged discharge pipes are not a serious problem at this time, they should be checked periodically (semi-annual) to see if additional pipes become closed which would warrant the cost of correction.

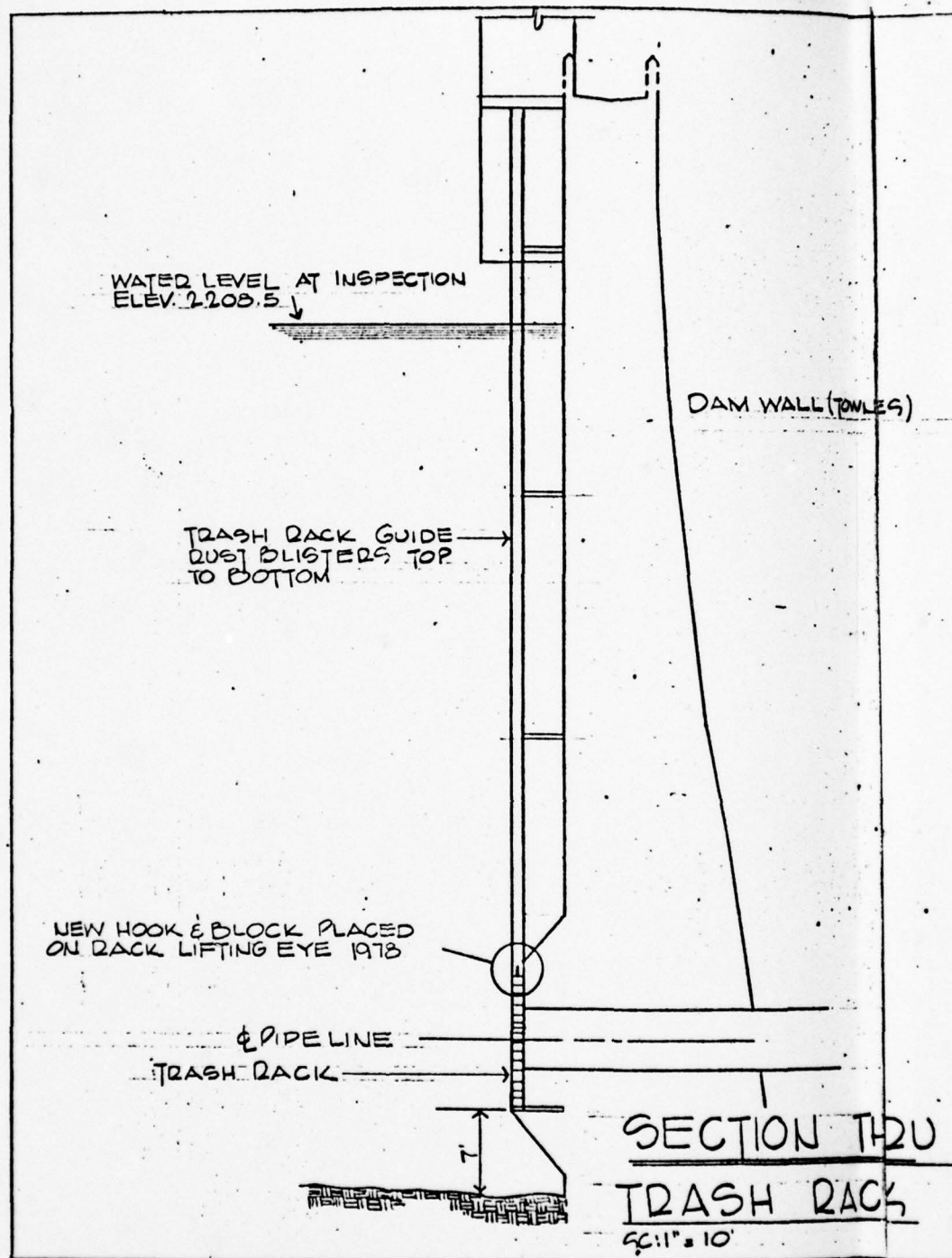
We also recommend semi-annual opening and closing of the head gate and downstream valves to assure their continued functions.

RECOMMENDATIONS FOR TOWNES DAM

(Following replacement of the trash rack lifting cable, we believe the system to be in good condition.

We do recommend cleaning of the trash rack guides prior to any needs to raise and lower the trash rack.

We also recommend semi-annual operation of downstream valve to assure to continued functions.



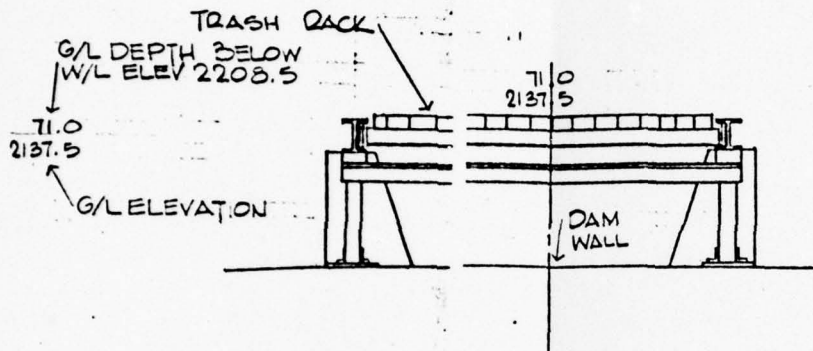
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68.0
2140.5

72.8
2135.7

71.5
2137.0

DAM WALL (TOWNS)



PLAN AT
TRASH RACK
SC 1/4" = 1'-0"
TOWNS DAM

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SECTION THRU
TRASH RACK

10'



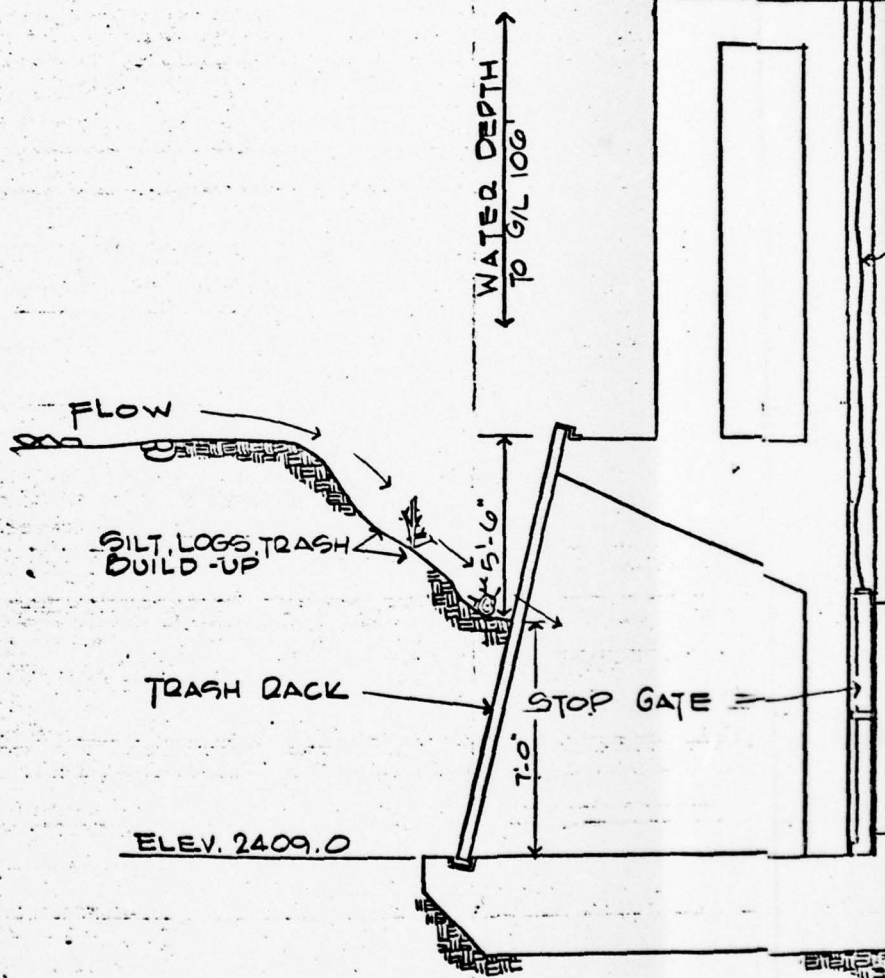
BY
LOGAN DIVING, INC.
JACKSONVILLE, FLA. 32207

DWG 2 OF 2

FIGURE

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WATER LEVEL AT INSPECTION
ELEV. 2521.75



SECTION THRU TRASH RACK

SCALE 1/4" = 1'-0"

TALBOTT DAM

BY PRACTICABLE
DDG

2. LEVEL AT INSPECTION
2521.75

WATER DEPTH
TO G/L 106'

CABLE TO
CONTROL HOUSE

STOP GATE

7'-0"

ION THRU
H RACK

1/4" = 1'-0"

DAM

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UNDERWATER INVESTIGATION		DRAWN BY J.W.H.
PINNACLES HYDRO-ELECTRIC PROJ. UPSTREAM INSPECTION		CHECKED BY JGM
TALBOTT & TOWNES DAMS FOR		DATE APR 78
DANVILLE ELECTRIC DEPT.		SCALE HOR. 1/4" = 1'-0" VER. 1/8" = 1'-0"
BY LOGAN DIVING, INC. JACKSONVILLE, FLA. 32207		DRWG. NO. 1 of 2

APPENDIX VI
CONTRACT SPECIFICATIONS
(SELECTED PAGES)

SPECIFICATION
for
for

BIG BEND DAM

I. GENERAL

(a) Purpose

These specifications and accompanying contract documents, including the drawings and schedules are intended to cover the clearing of the Big Bend Reservoir and the building of the Big Bend Dam, both of which are a part of the Pinnacles Hydro-Electric Project. The Contractor shall do all of the work indicated in these specifications and on the plans and such additional work as may be necessary to complete the dam and appurtenances in a substantial and acceptable manner and leave the work in a neat and finished condition.

(b) Description

The Big Bend Dam will be constructed across the Dan River about 2 miles below its confluence with Big Ivy Creek. The dam is located about 4 miles south of Meadows of Dan in Patrick County, Virginia. Meadows of Dan is 16 miles west on Route 58 from Stuart, Virginia, the terminal of the Danville and Western Railway, and 34 miles east on Route 58 from Galax, Virginia, on the Norfolk and Western Railway. Both railroads have storage tracks and unloading space which can be made available to the Contractors. From Meadows of Dan an improved road leads 6-1/2 miles on an easy grade to the site of the Big Bend Dam.

The width of the Dan River at the dam site is about 50 feet and the depth about 18 inches. The average flow is approximately 85 second feet and the estimated maximum monthly flow in the past 15 years has been about 300 second feet, the flood water occurring either in the Spring or in the Fall of the year. It is believed that a small coffer dam and flume leading across the dam site would be sufficient to unwater the site preliminary to excavating for the foundation.

The dam is a massive concrete arch dam built in sections, or blocks about 40 feet long. The dam is about 140 ft. high and 510 ft. long.

The outlet will be built in the center of one of these blocks and consists of a 36" steel pipe through the dam controlled by an electrically operated 36" valve. On the upstream side of the outlet, a screen will be provided. At the downstream side of the outlet a valve house is to be built to house the valve and its operating mechanism. At the crest of the dam a walkway is provided with stairways at each abutment.

trees, bushes and other perishable matter. Trees shall be cut off not higher than 18 inches above the ground. All material so removed shall be piled in windrows and burned. The burning shall be done at such time and in such manner as may be required by the Engineer. Any available timber within the area to be flooded by the reservoir may be cut and used by the Contractor for construction purposes.

(b) Measurement and Payment

Measurement and payment for reservoir clearing shall be made on the unit prices bid in the schedule. It is stipulated and agreed that the planimeter shall be considered an instrument of precision adapted to the measurement of cleared areas.

VII. EXCAVATION

(a) Classification

Except as otherwise provided in these specifications, all material removed from excavation will be measured in excavation only to the neat lines shown on the drawings or prescribed by the Engineer and will be classified as follows:

Rock Excavation

All solid rock in place which cannot be removed until loosened by blasting, barring or wedging, and all boulders or detached pieces of solid rock more than 1/2 cubic yard in volume. No material except boulders and detached pieces of solid rock will be classed as rock excavation which is not actually loosened by blasting before removal, unless blasting is prohibited, and barring, wedging or similar methods are prescribed by the Engineer.

Earth Excavation

All soft overburden not including disintegrated rock which can be removed by sluicing or with excavating machinery.

No additional allowance over the unit prices bid for excavation will be made on account of any of the material being wet or frozen.

(b) Blasting

Blasting will be permitted only when proper precautions are taken for the protection of persons, the work and private property and any damage done to the work or to property shall be repaired by the Contractor at the Contractor's expense. Exploders shall in no case be stored, transported or kept in the same place in which dynamite or other explosives are stored, transported or kept. In general the precautions taken to prevent accidents shall be subject to the approval of the Engineer, but the Contractor shall be liable for all injuries to, or death of persons, or damage to property caused by blasting or explosives. Electric blasting machines shall be used on all work.

(c) Excavation for Foundations

The excavation for the dam foundations shall be made to a sufficient depth to secure foundation on sound ledge rock, free from open seams or other objectionable features. Unusual precaution shall be taken to preserve the rock outside of the line of excavation in the soundest possible condition. Blasting may be done only to the extent directed by the Engineer with explosives of such moderate power and in such locations as will neither crack nor damage the rock outside of the prescribed excavation limits. Whenever in the opinion of the Engineer, blasting is likely to injure the rock upon or across which concrete is to be placed, the use of explosives shall be discontinued and the excavation completed by wedging, barring and picking, or other suitable methods. The excavation shall be roughly shaped, as directed by the Engineer to approximately horizontal steps, at least 5 feet high and separated by approximately vertical radial planes. Special care shall be taken in excavating these steps to avoid shattering or damaging the adjacent rock.

(d) Limits of Foundation Excavation

Earth and loose rock shall be excavated on the steepest practical slopes with the following modification. Where soft overburden is encountered the slopes shall not be less than 1:1 and the toe of the slopes shall not be less than 30" from the beginning of the rock cut. Where rock is encountered the slopes shall not be less than 1/4:1. Solid rock shall be excavated as close as practical to the neat lines of the structure, except for the protection of the upstream face of the dam the minimum excavation shall consist of a 5 ft. ledge along the entire face of the dam.

The drawings refer to, and show "sound rock" contours and elevations determined from diamond drill core borings. These contours and elevations were used in the design of the dam. The actual sound rock elevation will be determined by the Engineer after an inspection of excavation. If the approved sound rock elevation so determined differs more than 5 ft. from the sound rock elevation and contours shown on the contract drawings it may be necessary to redesign the dam. The Contractor shall have no right for claims due to delay that may be necessary to make changes in the design.

(e) Disposal of Materials

At the upstream face of the dam no excavated material shall be disposed of in a distance of 500 ft. and the trench along the upstream face shall not be backfilled except as noted. At the downstream side of the dam backfill shall be provided and grading shall be done in accordance with the drawings.

Excavated material not used for backfill shall be disposed of in a manner and at locations approved by the Engineer. All spoil banks shall be located where they will not interfere with the natural flow of the river or with the discharge from the spillway. Spoil banks shall be

located where they will not detract from the appearance of the structure or interfere with the accessibility of the structure for operation. Spoil banks shall be leveled and trimmed to reasonably regular lines and the Contractor shall not be entitled to any additional compensation on account of such refinement. All combustible material such as trees, logs, brush or roots required to be removed from the excavation for the dam or other works, or from the sites of the spoil banks or otherwise shall be piled and burned under the direction of the Engineer. The cost of disposing of all excavated material that is wasted and all other work described in this paragraph shall be included in the price bid in the schedule for excavation.

In general the manner of disposing of materials is indicated on drawing 1257-707, which shows two spoil banks on either side of the river extending from the downstream face of the dam a distance of approximately 350 ft. downstream.

The top of these spoil banks is at elevation 2450. and the toes of the banks are 40 ft. apart on either side of the river, leaving a straight channel for water passing over the spillway.

(f) Measurement and Payment

Measurement and payment will be made to the prescribed neat lines of the excavation and at the unit prices bid in the schedule. Backfill will be measured for payment in place and paid for at the price bid in the schedule. Any excavation outside of the limits prescribed which is required to be backfilled shall be done by and at the expense of the Contractor.

VIII. FOUNDATION PREPARATION

(a) Description

After the sound rock elevation has been determined and the minimum excavation required on the upstream face of the dam has been completed the base rock shall be prepared for the building of the dam.

The surface of the rock shall be left rough so as to bond well with the concrete, and where necessary shall be cut to rough benches or steps as directed by the Engineer to secure the required roughness. Care must be taken not to shatter or disturb rock foundations unnecessarily. All loose fragments, dirt and spalls must be removed from the rock surface before concrete is poured. Wire brooms, hammers, picks or streams of water, air or steam or other effective means shall be used to clean the foundation.

(b) Grouting Base Rock

The base rock underneath the dam shall be prepared by thorough cement grouting. The purpose of this grout is to provide a water tight curtain below the dam along its upstream face and also to provide a uni-

form and consolidated foundation for the dam. While the general scheme of grouting to be employed is shown on the drawings, the depth and location of borings is illustrative only, and may be different when the excavation is made and the exploration holes are drilled.

Approximately 10% of the total number of grout holes shall be drilled with diamond core drills, 1-1/2" core. The location of these exploration holes will be determined by the Engineer, and the record of the holes and the cores shall be kept and the cores properly preserved. Each of the diamond drill borings shall extend a minimum of 30 feet below the sound rock elevation and if the last 10 feet of the hole does not establish a satisfactory rock, the hole shall be extended up to a depth of 50 feet.

After a study of the cores of the exploration holes has been made, grout holes shall be drilled on such spacing and to such depth as the Engineer may determine. All grout holes shall be drilled with jack hammers, or by any other method, and shall have a minimum diameter of 1-1/2".

Immediately preceding grouting, each hole shall be thoroughly cleaned by lowering a 1 inch pipe to the bottom of the hole and pumping water through the pipe until a uniform flow of clean water is returned from the hole. After such flow is established the pumping of water shall be continued at least 15 minutes, at the end of which time the hole shall be dried with compressed air, the pipe carefully removed and the hole filled with grout, plugged, and connected to the grout pump which shall be kept pumping until a pressure of 100 $\frac{1}{2}$ per square inch is established. The pressure shall be maintained at 100 $\frac{1}{2}$ until the hole refuses to take additional grout and for 15 minutes longer. If this cannot be accomplished, additional holes shall be drilled in the vicinity of the unsatisfactory hole and all shall be grouted in the same manner.

Holes shall be drilled and grouted in the sequence shown on the drawings. A complete record shall be kept of the time each hole was drilled, washed and grouted, and also the amount of grout admitted during each 10 minute interval shall be recorded.

Grout for the base rock shall be composed of cement and water, or cement, sand and water in proportions to be determined by the Engineer. Sand shall be clean and of such fineness that 100% will pass a screen with 64 openings per square inch and 50% will pass a screen with 1600 openings per square inch. The apparatus for mixing and placing the grout shall be of a type approved by the Engineer and capable of mixing and stirring the grout and forcing it continually into holes at any desired pressure up to 100 lbs. per square inch. If during the grouting of any grout hole, grout is found to flow from adjacent holes in sufficient quantity to seriously interfere with the grouting operation, or to cause loss of grout such holes shall be temporarily capped. Where such capping is not essential ungrouted holes shall be left open to facilitate the escape of air and water as the grout is forced in.

Grouting of the base will be paid for at the unit price bid in the schedule and shall include the cost of labor, material, plant and operations incident to the grouting. materials for grout will be measured dry as they are placed in the grouting machine and payment will be made for each batch or fractional batch actually forced into the holes. No payment will be made for grout lost or rejected.

after the dam is poured holes shall be drilled approximately 10 ft. on centers along the upstream face so as to intersect the joints for the concrete and the rock at the base and at the vertical faces of the steps, and penetrate the rock at least 3 ft. These holes shall also be grouted under 100 lbs. of pressure.

IX. CONCRETE

(a) Description

Since there will be about 30,000 cubic yards in the dam and related structures, the cost of the concrete aggregate is a very considerable factor in the total cost of the job. The Field Engineer for the City of Danville has made a complete survey of the sources of aggregate in the vicinity of the work. The results of this survey and of the tests of concrete made from these materials is available in a report on file in the Field Office of the Project at Meadows of Dan, Va.

The more important acceptable sources of both coarse and fine aggregate which are available in sufficient quantities in the vicinity of the work are summarized as follows:

Coarse Aggregate

1. Quarry site explored by diamond drill boring and under option to the City of Danville, located on Route 58, 9 miles west of Cruise's Store, Meadows of Dan. Overburden about 30 feet. Both fine and coarse aggregate can be made from this granite. Haul 16 miles to the dam site.
2. Dolomite from the Radford Limestone Company, Radford, Va., shipped to Galax, and hauled 40 miles to the dam site.

Fine Aggregate

1. Crushed granite from Quarry Site on Route 58, 14 miles haul to dam site.
2. Crushed dolomite from Radford quarry, delivered at site.
3. Petersburg Sand shipped to Stuart; haul 22 miles.
4. Suitable fine aggregate can be made by crushing white flint stone scattered over the fields in the vicinity of the work.

The Contractor, in his bid, shall state the source of both fine and coarse aggregate upon which his bid price on concrete is based.

Appended to these specifications is a general specification for concrete defining the minimum quantities of cement to be used for the two classes of concrete. The "concrete" specifications are however, subject to such modification of the proportions of sand and stone but not of cement as will produce the most economical concrete possible, having an ultimate strength at the age of 28 days of not less than 3000# per sq. in. for Class "A" concrete and not less than 2000# for Class "B" concrete. The exact proportions to be used for the different parts of the work will be determined by the Field Engineer after analysis and tests have been made of accepted aggregates furnished by the Contractor. The Field Engineer shall at frequent intervals make such additional tests and analysis of concrete materials and the resulting concrete and such changes in the proportions as may be necessary to secure the required economy, and the Contractor shall be entitled to no additional compensation because of such changes.

(b) Use of Vibrator

In addition to the strength requirement it is essential that the concrete be uniformly dense and free from segregation and honeycomb. Only sufficient water shall be used to secure suitable workability as determined by the Engineer to flow properly into place with thorough spading or working. In general, a consistency corresponding to a slump, not greater than 3" shall be used for Class "A" concrete and not greater than 6" for Class "B" concrete. If it appears after test that the concrete materials are such that dense concrete cannot be formed without the aid of vibrators, the specified slump shall be reduced so that when vibrated a minimum flow will be secured. If vibrators are used, they shall be of the mechanical type approved by the Engineer and shall have a frequency of not less than 6000 RPM under load, with an effective vibrating radius such that insertions are required not less than 12" apart.

All concrete to be vibrated shall be left for a minimum period of 15 minutes before vibrating is commenced. The concrete shall also be placed continuously and in such areas and in such layers of thickness as may be determined by the Engineer. When sufficient time has elapsed to allow vibrating to begin, the operator shall be instructed in the use of the vibrator, which method shall be maintained at all times. Vibrating shall be applied only inside the concrete batch and shall not come in contact with the forms.

(c) Classification

Class "A", 3000# concrete shall be used in the apron and the main body of the dam up to elevation 2538.5. Class "B", 2000# concrete shall be used for the stairways and intake structure and valve house and the parapet and walkway above elevation 2540.5, and instrument pier.

(d) Forms

It is suggested that the Contractor use panel forms for the main body of the dam with suitable devices to hold adjacent ends and edges of panels tight and in accurate alignment. All forms shall be

(e) Measurement and Payment

Concrete in the main dam and apron will be measured by the cubic yards actually placed and paid for at the unit price bid in the schedule for Class A concrete, which price shall include all labor and all material excepting reinforcement.

Concrete in parapets and stairway will be measured by the cubic yards actually placed and paid for at the unit price bid in the schedule for Class B concrete, which price shall include all material and all labor excepting reinforcement.

The valve house and intake structure will be paid for at the lump sum price bid in the schedule for these items.

(f) Vertical Construction Joints

Each vertical construction joint shall be provided with keyways, grout stops, grouting slots, and grouting pipes as shown on the drawings. The grouting slots are provided in the concrete block in which the concrete is placed first, by nailing a wooden strip to the forms and providing pipe connections and nails as shown. After the forms are stripped, the grouting slot is protected by a sheet iron cover fastened bending the nails as shown on the drawings. When the concrete is placed in the adjoining blocks the grouting slots are to be kept clean by blowing compressed air through them, connecting the compressed air supply individually to each grout pipe at the bottom of the block. After the dam is finished, the grout pipes at the bottom of the different blocks are connected by a header immediately preceding the grouting.

(g) Grouting Construction Joints

The grouting of the joints is to be done by the use of two grouting pumps. The grouting shall be done at two symmetrically located vertical joints, starting with the two joints at the extreme ends of the dam and moving toward the center, always grouting two symmetrically located joints simultaneously. After the installation of the pumps at two joints, water should be pumped through the grout pipes until all air is removed and a uniform flow of water established through the outgoing pipe, after which the water should be pumped through the joint for at least one hour. After water has been pumped through the joint for one hour, without stopping the pump and without interrupting the flow, grout should be gradually admitted in the pump and the pumping continued until a uniform flow of grout will be established through the outlet pipe. Then the outlet pipe should be plugged by use of a vent provided on this pipe, the pump working continuously, increasing the pressure in the grouting system until the specified pressure of 100 lbs. per sq. in. is obtained. The pump should be kept pumping under this pressure until the joint refuses to admit any grout and one half hour afterwards.

If, during the grouting operation the pressure cannot be built up, due to leakage, the whole procedure should be reversed, water being admitted in the grouting pipes and pumped until all grout is washed out. Afterwards the leak must be found and stopped and the whole grouting procedure repeated until the joint is satisfactorily grouted under

the whole specified pressure of 100 lbs. per sq. in.

Grout for vertical construction joints shall be composed of cement and water in proportions to be determined by the Engineer. Grouting will be paid for at the unit price bid in the schedule and shall include all labor, material, plant and incidentals necessary to the operation, including metal grout stops and plates over V slots. Measurement of dry materials will be made at the grouting pump and payment will be made for each full batch or fractional batch actually forced into the vertical construction joints. No payment will be made for lost, wasted or rejected grout.

XI. STRUCTURAL STEEL

(a) Description

The principal structural steel items are the trash rack, its guides and supports and the stop log slot at the upstream face of the dam; the ladder, pipe railing and miscellaneous items in the valve house.

(b) Materials and Workmanship

All structural steel shall conform to the latest specification of the American Society for Testing Materials for structural steel for buildings. Insofar as applicable, the design, fabrication, and erection shall be in accordance with the standard specifications of the American Society of Steel Construction for structural steel for buildings. All workmanship shall be equal to the best practice in the modern structural shops. The several pieces forming built up sections shall be straight and fit closely together and the finished members shall be free from twists, bends or open joints. At the option of the Contractor shop connections shall be electric welded.

(c) Drawings

The Contractor shall submit 3 sets of shop drawings to the Engineers for approval before proceeding with the work.

(d) Erection

The steel work in general shall be knocked down in pieces not over 25 feet long. All structural steel work shall be accurately set and properly secured in place. All field connections will be riveted as promptly as possible as the erection progresses.

(e) Anchor Bolts and Anchorages

Anchor bolts and anchorages shall be built into the connected work in advance. Particular care shall be taken to see that the alignment of the trash rack guides will permit of raising and lowering the trash rack without binding.

(c) Conveying

Concrete shall be conveyed from mixer forms as rapidly as practicable by methods which will prevent segregation or loss of ingredients. It shall be deposited as nearly as practicable in its final position. Chutes used shall be such that the concrete slides in them and does not flow. Chutes with a flatter slope than 1 on 2 will not be permitted. There shall be no free vertical drop greater than 5 feet.

(d) Placing Concrete on Rock Foundation.

Rock surfaces shall be approximately horizontal or stepped, rough and free from loose material or other matter interfering with satisfactory bond. The rock shall be washed and scrubbed with steel brushes or brooms and covered with a layer of cement mortar about one-half (1/2) inch thick immediately ahead of the placing of the base course of concrete. The mortar shall be of the same sand-cement ratio as that used in the concrete. Rock dowel bars shall be grouted into place and left undisturbed for a period of at least seventy-two (72) hours in advance of placing concrete.

(e) Seepage Water.

Adequate measures shall be taken by the Contractor to control the ground water table to an elevation sufficiently below the base of the structure footings to prevent - First: The contact of seepage water with the green concrete as it is being placed, and Second: the formation of uplift pressure against the newly placed concrete before full set has taken place. When and if the conditions envisaged in this paragraph are encountered on the work, the seepage water control measures taken and the condition of the footing base shall have the approval of the Engineer before any base course concrete is placed.

(f) Condition of Concrete.

All concrete shall be placed before initial set has occurred, and in no event after it has contained its water content for more than thirty (30) minutes. Concrete rejected on account of the foregoing stipulation shall not be used as "plums" or broken up for use as aggregate in subsequent mixes. Unless otherwise specified, all concrete shall be placed in the dry upon clean, damp surfaces, free from excess water, and never upon soft mud, dry porous earth, or frozen earth, or upon filled-in material that has not been subjected to approved puddling and/or tamping so that ultimate settlement has occurred.

(g) Courses, Lifts, and Sections for Pouring.

Concrete throughout the entire structure shall be poured in horizontal lifts and sections, and construction joints shall be placed at the elevations shown on the plans or as otherwise approved and/or directed by the Engineer.

(h) Working Concrete in Forms.

The concrete shall be compacted and worked in an approved manner into all corners and angles of the forms and around reinforcement and embedded fixtures. Where required by the Engineer, special pneumatic or electric vibrating or tamping devices shall be used. Care shall be taken in dropping concrete through reinforcement so that no segregation of the coarse aggregate occurs. On flat surfaces where the congestion of steel near the forms makes placing difficult, a mortar of the same cement-sand ratio as is used in the concrete shall be first deposited to cover the forms. All top surfaces shall be carried slightly above the forms and struck off by board finish when the water settlement has taken place. Care shall be taken to see that all excess water is forced out and removed in making this finish.

(i) Construction Joint Details.

Vertical joints shall be formed with tongue-and-groove keys and dowel bars as shown on the plans or as directed by the Engineer. Horizontal joints shall be formed with keys; or where horizontal pressure is always in one direction, with steps. Dowel bars shall be placed as shown on plans and where directed by the Engineer in the various courses at construction joint levels. These bars shall be placed in the concrete before the concrete takes its initial set and shall not be bent, lined, or disturbed until the concrete has thoroughly set up. All concrete in an underlying course shall have been in place not less than 8 hours before concrete in upper course resting thereon is placed. Before placing is resumed all excess water and laitance shall be removed and the concrete shall be cut away, where necessary, to insure a strong dense concrete at the joint. Where necessary to secure adequate bond, the surface of existing concrete shall be cleaned and roughened and then shall be spread with a 1/2 inch layer of mortar of the same cement-sand ratio as used in the concrete immediately before the new concrete is deposited.

(j) Cold Weather.

Concrete shall not be placed when either the ambient atmospheric temperature is below 35 degrees F., or if in the opinion of the Engineer the concrete is likely to be subjected to freezing temperature before final set has occurred, unless specifically authorized. When so authorized, the materials shall be heated so that the temperature of the concrete when deposited shall be not less than 50 degrees F., nor more than 100 degrees F. Salt or other chemicals shall not be admitted into the mixture to prevent freezing.

Cement Grout

Cement grout used for filling rock test holes, rock dowel holes and anchor bolt holes shall consist of one part cement and one part sand with only sufficient water added to make a thick workable mortar.

APPENDIX VII

REFERENCES

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REFERENCES

1. HEC-1DB Flood Hydrograph Package, (Hydrologic Engineering Center, U. S. Army Corps of Engineers, July 1978).
2. "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian," Hydrometeorological Report No. 33, (U. S. Weather Bureau, April 1956).
3. "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, (U. S. Weather Bureau, May 1961).